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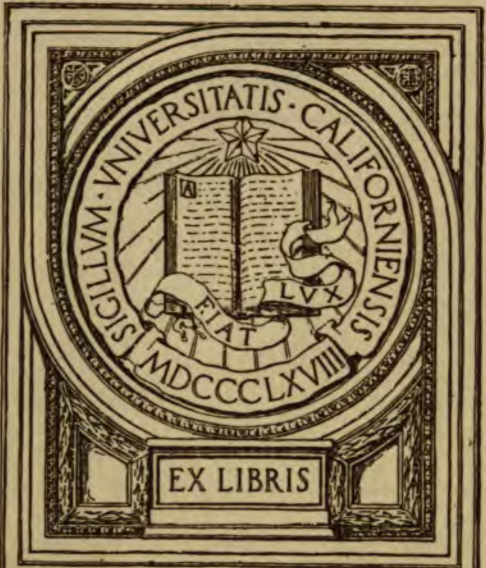

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GENERAL

THE

DETAILED DESIGN

OF A

RAILROAD BRIDGE

BY

WM. H. BURR, C.E.

AND

MYRON S. FALK, C.E.

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THE DETAILED DESIGN OF A RAILROAD BRIDGE.

BY WM. H. BURR, C. E., AND MYRON S. FALK, C. E.

The design of iron and steel truss bridges must necessarily contain little that is new; but on the other hand, such rapid development has been made even in so ordinary a piece of engineering work as the design of a steel bridge that a full and systematic statement of it as at present practiced must be of distinct value. While, therefore, the following exposition of the design of a steel railroad truss bridge cannot be regarded as disclosing much that partakes of the nature of an investigation of a new subject, it will at least exhibit a systematic application of engineering science in an important field of engineering practice.

The design of the bridge will be based on the following data, which in engineering practice would be furnished by the railroad company for which the bridge is to be built.

Length of span between centers of end pins. Panel length, 29′ 2″, and number of panels, 12	350′	0″
Distance between centers of trusses	18′	0″
Distance between centers of stringers	7′	6″
The center height U_5L_5 is	63′	0″
The intermediate height U_3L_3 is	54′	0″
The end height U_1L_1	36′	0″

The form of truss is of the type known as the broken upper chord with subdivided panels, as is clearly shown on the stress sheet, Plate I. As specified by the railroad company all of the material of the bridge is to be of medium steel; all the rivets are to be ⅞ inch, except that ¾-inch rivets may be used in the flanges of 12-inch channel posts, in horizontal struts at the middle of posts and in the overhead lateral or sway bracing. The structure is to be designed according to Cooper's specifications of 1896.

ART. I. STRESSES IN THE STRUCTURE.

The stresses in the structure are caused by its own or dead weight and by the live or moving load. The dead load may be divided into two parts, the weight of the floor system consisting of rails, ties, track fastenings, stringers and floor beams, and the trusses and bracing. Excerpts from Cooper's specifications relating to these loadings are the following:

§ 23. All the structures shall be proportioned to carry the following loads:

1. The weight of metal in the structure.

2. A floor weighing 400 pounds per linear foot of track to consist of rails, ties and guard timbers only.

These two items, taken together, shall constitute the dead load.

*3. A "live load" on each track, supposed to be moving in either direction, consisting of 2 "consolidation" engines, coupled and followed by a train load, distributed as shown on diagram E 40 (or 100,000 pounds equally distributed on two pairs of driving wheels, spaced seven and a half feet, center to center).**

The maximum stresses due to all positions of either of the above "live loads" of the required class, and of the "dead load," shall be taken to proportion all parts of the structure.

§ 27. Variation in temperature, to the extent of 150° F., shall be provided for.

§ 28. When the structures are on curves, the additional effects due to the centrifugal force of trains shall be considered as live load. It will be assumed to act 5 feet above base of rail, and will be computed for a speed of $50 \times 2d$ *miles per hour;* d *being the degree of curve.*

§ 29. All parts shall be so designed that the stresses coming upon them can be accurately calculated.

§ 28 does not apply to the structure under consideration, and the requirements of § 27 may be fulfilled by placing one end of the bridge upon a nest of movable rollers.

ART. 2. DETERMINATION OF DEAD LOAD STRESSES.

The weight per linear foot of the trusses, not including the floor system but including all the lateral and cross-bracing, may be approximately estimated by means of the following formula: $W = al$, where W is the weight per linear foot of both trusses and bracing in pounds, a is a numerical coefficient and l is the length of span in feet. In the bridge under consideration a may be taken as 7.75. The weight of the two trusses then becomes 2,700 pounds per linear foot, or 39,380 pounds per panel per truss. This weight must be equally divided between the upper and lower panel points. The specifications require the weight of track to be taken at 400 pounds per linear foot, while the remainder of the floor system may be assumed to weigh 500 pounds per linear foot; hence these loads become 5,830 and 7,290 pounds per panel per truss respectively. They act at the lower panel points.

---

* This part of the specifications is usually employed in determining the maximum bending moments and shears in the floor system In the example under consideration this alternative loading has been neglected

The upper panel point loading is then 19,690 pounds but the lower panel point loading becomes 32,810 pounds.

The dead load stresses are easily determined by graphic methods

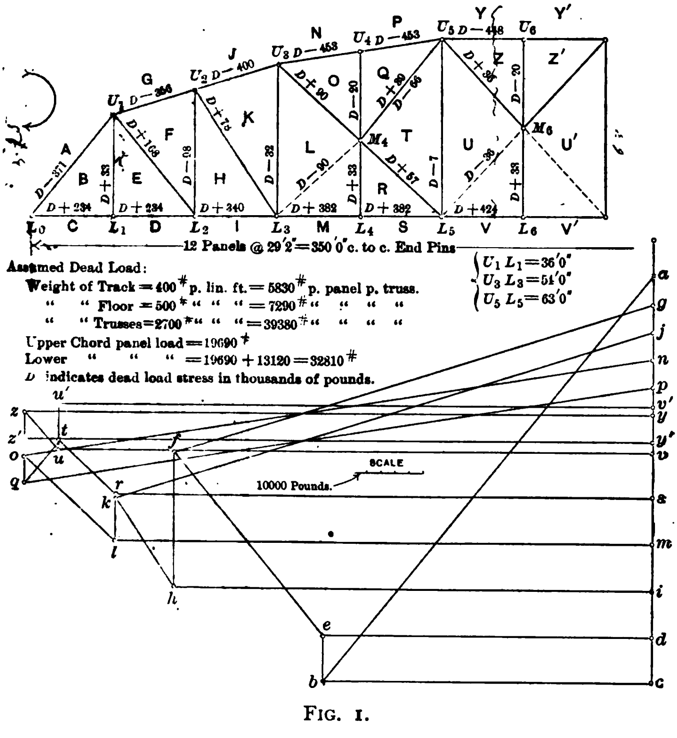

FIG. 1.

as shown in Fig. 1; for that figure only those web members shown in full lines are assumed to act, the dotted members acting as counters. It is unnecessary to explain the details of the figure; but to insure the accuracy of the graphic diagram it is important to check analytically the last stress found; the stress in L_5L_6 for instance is so found by passing a section through U_5U_6, U_5M_6 and L_5L_6 and taking the center at U_5.

The compression stresses in L_3M_4 and M_4U_5 are found by assuming U_3M_4 not to exist and are quickly determined by the method of sections.

ART. 3. DETERMINATION OF LIVE LOAD STRESSES IN THE CHORDS.

The position of moving load for the greatest bending moment at any point in a truss or beam is found by the following familiar criterion, it being supposed for convenience that the train passes along the truss from right to left with W_1 as the front load:

$$\frac{l'}{l} = \frac{W_1 + W_2 + W_3 \cdots W_n'}{W_1 + W_2 + W_3 \cdots W_n}. \qquad (1)$$

In Eq. 1, l' represents the distance from the left end of the truss to the center of moments, *i. e.*, to the point in question, and l, the total length of the truss; $W_1 + W_2 \ldots W_n'$ the sum of the loads on l' and $W_1 + W_2 \ldots W_n$ the sum of the loads on l.

By the method of sections the stress in a chord member is found by dividing the maximum bending moment at the proper panel point by the normal distance (or lever arm) from the panel point to the chord member in question. This method is directly applicable to the truss cousidered, although a modification * is necessary in case of subdivided panels.

By drawing a reaction influence line both the truss reaction and the bending moment at any point for any position of the moving load may be at once measured; the diagram should be made on a large scale in order to attain the highest practicable degree of accuracy.

Number.	Wheel Load Number.	At Panel Point.	Stress in Pounds.	Remarks.
L_0L_1	5	L_1	+333,000	Train advancing from right to left.
L_1L_2	5	L_1	+333,000	"
L_2L_3	13	L_2	+480,000	"
L_3L_4	17	L_3	+537,000	"
L_4L_5	17	L_3	+537,000	"
L_5L_6	23	L_5	+590,000	'
L_0U_1	5	L_1	—528,000	'
U_1U_2	13	L_2	—502,000	"
U_2U_3	17	L_3	—562,000	"
U_3U_4	20	L_4	—631,000	Criterion of the subdivided panel.
U_4U_5	20	L_4	—631,000	
U_5U_6	26	L_6	—624,000	

---

* See paper "On the Use of Influence Lines in Graphic Statics," SCHOOL OF MINES QUARTERLY, July, 1903, p. 452.

By applying the criterion, Eq. 1, and completing these diagrams, the results shown in the preceding table will be obtained. The table exhibits both the positions of the concentrations for the greatest bending moments and the greatest stresses in the various chord members.

The uniform load is treated as a uniform series of loads of 25,000 pounds each; *i. e.*, load No. 23 in the table indicates the fifth load of this series.

ART. 4. DETERMINATION OF LIVE LOAD STRESSES IN WEB MEMBERS.

The graphic methods explained in the April, 1903, number of the SCHOOL OF MINES QUARTERLY were used to determine the position of the loading causing the maximum stresses in the web members of this truss, the modifications necessary for subdivided panels being given in the July number of the same publication.

The application of those methods gives all the results, both as to positions of loading and the greatest web stresses, exhibited in the following table:

Member.	Wheel Load Number.	At Panel Point.	Stress in Pounds.	Remarks.
U_1L_2	4	L_2	+257,000	Train advancing from right to left.
U_2L_2	3	L_3	−161,000	"
U_2L_3	3	L_3	+160,000	"
U_3L_3	3	L_4	−103,000	"
U_3M_4	3	L_4, for panel L_3L_4	+204,000	"
M_4L_5	5	L_5, for panel L_3L_5	+168,000	"
U_5M_4	When L_4M_4 is maximum		+ 62,000	"
	3	L_3, for panel L_4L_5	+104,000	Train advancing from left to right.
M_4L_3	5	L_3, for panel L_3L_5	+ 82,000	"
U_5L_5	3	L_6	− 79,000	Train advancing from right to left.
U_5M_6	3	L_6, for panel L_5L_6	+173,000	"
M_6L_5	3	L_5, for panel L_5L_5	+136,000	"

The members U_1L_1, M_4L_4 and M_6L_6 are simply hangers. They carry in tension the maximum floor beam load, which occurs when wheel No. 5 is directly at the hanger.

The members U_4M_4 and U_6M_6 are simply supporting struts; they carry in compression the dead load assumed to exist at their upper ends, but they carry no live load.

The horizontal struts M_3M_4 and M_5M_6 are intended simply to prevent flexure of the long column into which they frame. They are subject to no direct stresses of either tension or compression

ART. 5. DETERMINATION OF WIND STRESSES.

Cooper's specifications prescribe:

"§ *24. To provide for wind stresses and vibrations * * * the bottom lateral bracing in through bridges, for all spans up to 300 feet, shall be proportioned to resist a lateral force of 450 pounds for each foot of the span; 300 pounds of this to be treated as a moving load, and as acting on a train of cars, at a line 8.5 feet above base of rail.*

"** * * the top lateral bracing in through bridges for all spans up to 300 feet shall be proportioned to resist a lateral force of 150 pounds for each foot of span.*

"*For spans exceeding 300 feet add in each of the above cases 10 pounds for each additional 30 feet of span.*

In accordance with these specifications, the assumed wind load for the bottom chord will be 470 pounds per lin. ft., and for the upper chord 170 pounds per lin. ft.

Upper Lateral Bracing.

The stresses for the upper chord lateral bracing are found in the most expeditious way by analytical methods.

The bracing may be considered a truss of 12 panels lying in a horizontal plane with its abutments at the feet of the end posts. The loading is fixed in position and equal to $29.1 \times 170 = 4{,}950$ pounds per panel, the reaction at the end post being 29,700 pounds. The secant of the angle of inclination between the web members and the perpendicular to the plane of the main truss is 1.9. The stresses in the web members are then found by multiplying the shear in the respective panels by this secant, and they are given on the stress sheet. It should be noted that in this treatment the panel of the portal is considered a part of the wind bracing, although the true lateral bracing in that panel is replaced by a portal frame. This involves no error in determining the stresses of the webbing of the system, and is on the side of safety for the chord members. The stresses in the chord members are easily found by the method of sections.

Lower Lateral Bracing.

The lower lateral bracing is designed for a moving load of 300 pounds per lineal foot and a fixed load of 170 pounds per lineal foot. The stresses in the chord members of that system are easily found by multiplying the stresses in the chord members of the upper lateral system by the ratio between the loads of the two systems, *i. e.*, $\frac{470}{170} = 2.76$. The resulting stresses are shown on the stress sheet.

The stresses in the webbing are found by taking the stationary load the same as for the upper chord bracing and combining its effects with those due to the moving load. The following examples illustrate the method employed:

Web Member in Panel L_0L_1.

The stress in this member has its maximum value when the entire truss is covered by the moving load. The reaction then becomes 52,500 pounds, and the shear in the panel is $52,500 - (29.1 \times 300) / 2 = 48,150$ pounds. The stress is then $48,150 \times 1.9 = 91,500$ pounds. To this must be added the stress due to the stationary wind load, viz., 5,100 pounds. The sum of these two quantities may then be assumed to be carried equally between the two intersecting web members within the panel, causing compression in the one and tension in the other member.

The resultant stress for each is therefore $\frac{1}{2}(91,500 + 51,000) = 71,250$ pounds.

Web Member in Panel L_1L_2.

For this member, $\frac{10}{11}$ of the bridge, or 318.1 feet, should be covered by the uniform moving load. The shear in the panel is then $43,400 - 3,620 = 39,780$ pounds, and the stress is 75,500 pounds. The stress due to the stationary load is 42,000 pounds; hence the final stress in the member is $\frac{1}{2}(75,500 + 42,000) = 58,750$ pounds.

The other web members of this lateral system are treated in precisely the same manner.

ART. 6. DESIGN OF LOWER CHORD MEMBERS.

The specifications which apply to the design of the lower chords are the following:

§ *30. All parts of the structures shall be proportioned in tension by the following allowed unit stresses:*

Bottom chords, main diagonals, counters and long vertical (forged eye-bars)—for live loads 10,000 pounds per square inch and for dead loads 20,000 pounds per square inch.

§ *36. The stresses in the truss members or trestle posts from the assumed wind forces need not be considered except as follows:*

1st. When the wind stresses on any member exceed one quarter of the maximum stresses due to the dead and live loads upon the same member. The section shall then be increased until the total stress per square inch will not exceed by more than one quarter the maximum fixed for dead and live loads only.

Member $L_5 - L_6$.

The stresses in this member, as taken from the stress sheet, are:

Dead load stress = + 424,000 pounds.
Live load stress = + 590,000 pounds.
Wind stress = ± 402,000 pounds.

Neglecting the wind stress, the area of cross-section required would be 80.2 sq. inches, as shown:

$$
\begin{aligned}
424{,}000 \div 20{,}000 &= 21.2 \text{ sq. inches.}\\
590{,}000 \div 10.000 &= \underline{59.0 \text{ sq. inches.}}\\
& 80.2 \text{ sq. inches.}
\end{aligned}
$$

The actual intensity of stress in the member would then be $(424{,}000 + 590{,}000) \div 80.2 \quad 12{,}600$ pounds per sq. inch. The unit stress due to wind would then be

$$\frac{402{,}000}{80.2} = 5{,}010 \text{ pounds,}$$

which exceeds by $5{,}010 - 3{,}150 = 1{,}860$ pounds, one quarter of the unit stress due to the dead and live load. The sectional area of the chord member, therefore, must be increased until the intensity of stress does not exceed $12{,}600 \times \frac{5}{4} = 15{,}750$ pounds. The combined dead, live and wind loads is 1,416,000 pounds. Hence the required area of cross-section will be

$$\frac{1{,}416{,}000}{15{,}750} = 89.9 \text{ sq. inches.}$$

The member will then be composed of 6 eye bars, each 8 inches deep by 1⅞ inches thick, the area of whose combined cross-section will be 90 sq. inches.

Other lower chord members are treated in exactly the same way, and it will suffice to consider one more only.

Member $L_3 - L_4$.

$$\begin{array}{rll} \text{Dead load stress} = +\ 382{,}000 \text{ lbs. @ } 20{,}000 \text{ lbs.} & = & 19.1 \text{ sq. in.} \\ \text{Live load stress} = +\ 537{,}000 \text{ lbs. @ } 10{,}000 \text{ lbs.} & = & \underline{53.7} \text{ sq. in.} \\ & & 72.8 \text{ sq. in.} \end{array}$$

$$\begin{array}{rl} \text{Wind stress} = + & \underline{356{,}000} \text{ lbs.} \\ \text{Total stress} = + & 1{,}275{,}000 \text{ lbs.} \end{array}$$

Neglecting wind stress, the average stress in the member would be

$$\frac{382{,}000 + 537{,}000}{72.8} = 12{,}620 \text{ pounds per sq. inch.}$$

Adding 25 per cent. to this stress on account of wind, the final section required becomes

$$\frac{1{,}275{,}000}{(12{,}620 + 3{,}154)} = 80.8 \text{ sq. inches.}$$

The member will be composed of 6 eye-bars, each $8 \times 1\frac{11}{16}$ ins., whose combined area will be 81 sq. ins.

The lower chord members in the first two panels of the truss are subjected to a more sudden loading than those in the other panels; it is therefore customary to form these members either of shapes suitable to resist compression, such as channels, or stiffen them by latticing the eye-bars together. In the present case the latter plan will be adopted; the two center eye-bars, four being necessary, will be tied together by $2\frac{1}{2} \times \frac{1}{2}$ in. lattice bars.

ART. 7. DESIGN OF UPPER CHORD MEMBERS.

The following specifications apply to the design of the upper chord:

§ 33. *Compression members shall be proportioned by the following allowed unit stresses:*

Chord segments.

$$P = 10{,}000 - 45\frac{l}{r} \text{ for live load stresses.}$$

$$P = 20{,}000 - 90\frac{l}{r} \text{ for dead load stresses.}$$

End posts are not to be considered chord segments. All posts of through bridges:

$$P = 8{,}500 - 45\frac{l}{r} \text{ for live load stresses.}$$

$$P = 17{,}000 - 90\frac{l}{r} \text{ for dead load stresses,}$$

in which expressions P is the allowed stress in pounds per square inch of cross-section, l is the length of compression member in inches; r is the least radius of gyration of the section in inches.

No compression member, however, shall have a length exceeding 125 times its least radius of gyration.

§ 70. The unsupported width of plates subjected to compression shall not exceed 30 times their thickness, except cover plates of top chords and end posts, which will be limited to 40 times their thickness.

§ 90. In compression chord sections the material must mostly be concentrated at the sides, in the angles and vertical ribs. Not more than one plate, and this not exceeding ½ inch in thickness, shall be used as a cover plate, except when necessary to resist bending stresses or to comply with § 70.

Member U_5U_6.

Dead load stress = — 448,000 pounds.
Live load stress = — 624,000 pounds.

It will now simplify the design work to reduce the dead load stress to the equivalent live load stress; for this member the live load stress, plus one half the dead load stress, becomes 848,000 pounds.

The allowed unit stress is not a constant quality for compression members, but it depends upon the radius of gyration of the cross-section of the member. It becomes necessary, therefore, in designing, to assume some trial value for the radius of gyration. In the rectangular or box form of section adopted for upper chord members, it will usually be found that the radius of gyration has a value of about 0.4 the depth of the side plates, the depth of the member being taken slightly greater than would be the case in a vertical column on account of the bending induced by its own weight. This depth is usually about one-twelfth to one-fifteenth the length.

In the case of U_5U_6, the depth of the member may be taken as 28 inches; the assumed radius of gyration becomes 11.2, and since l is 350 inches, the assumed unit stress becomes

$$P = 10,000 - 45\frac{350}{11.2} = 8,594 \text{ pounds per sq. inch.}$$

The approximate area required is therefore:

$$848,000 \div 8,594 = 98.7 \text{ sq. inches.}$$

A trial cross-section must now be formed from angles and plates of about this area, and tested to ascertain whether the assumed radius of gyration must be modified. The upper angles in the trial section should be made as light as possible, so that the cover plate may be largely balanced by the heavy bottom angles. Flat bars are frequently added to the horizontal legs of the lower angles for the same purpose. In this manner the center of gravity of the section may be brought down sufficiently near to mid depth to give the space needed inside the chord for the eye-bar heads, the pin axis being made to pass through the center of gravity of the section. At the same time there is gained the incidental but important advantage of an increased moment of inertia. If the chord were subject to no bending from its own weight, the axis of every pin should pass through the center of gravity of the section. It has been shown in Article 69,* that this flexure cannot be satisfactorily met and neutralized by the direct stress, particularly if

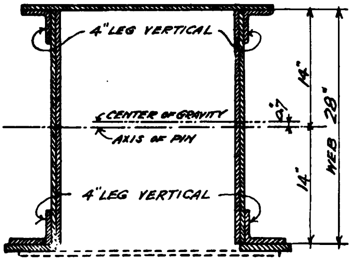

FIG. 2. Chord-section U_5-U_6.

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* Of Burr's "Course on Stresses in Bridges and Roof Trusses."

the chord is considered as continuous. It is best therefore to reduce the bending stresses by making the chord depth as great as possible and placing the axis of the pin either through the center of gravity of the section or but slightly below it.

Let the following section, Fig. 2, be tested:

		Sq. In.
1 cover plate	$30 \times \frac{5}{8}''$	= 18.75
2 L's	$4 \times 3\frac{1}{2} \times \frac{9}{16}''$	= 7.80
2 Webs	$28 \times \frac{1}{2}''$	= 28.00
2 "	$28 \times \frac{7}{16}''$	= 24.50
2 L's	$5 \times 4 \times \frac{9}{16}''$	= 9.50
2 bars	$6 \times \frac{7}{8}''$	= 10.50
		99.05

The position of the center of gravity of this section must first be determined by taking moments about the upper edge. The following calculation will determine its position.

Moment of cover plate	$18.75 \times 0.63 =$	11.8
" " upper angles	$7.80 \times 1.91 =$	14.9
" " web plates	$52.50 \times 14.63 =$	768.0
" " lower angles	$9.50 \times 27.51 =$	261.8
" " bars	$10.50 \times 29.07 =$	306.0
	Total =	1,362.5

Dividing this sum by the total area of the section determines the distance of the center of gravity from the upper edge, *i. e.*,

$$\frac{1362.5}{99.05} = 13.8 \text{ inches.}$$

The moment of inertia of the section must now be found about this center of gravity as an axis;

Cover plate,	$\frac{30 \times (.63)^3}{12} + [18.75 \times (13.5)^2] =$	3,420.6
Upper angles,	$2 \times 5.71 + [7.80 \times (11.9)^2] =$	1,114.4
Web plates,	$\frac{1.875 \times (28)^3}{12} + [52.50 \times (.8)^2] =$	3,463.6
Lower angles,	$2 \times 6.27 + [9.50 \times (13.7)^2] =$	1,794.5
Bars,	$\frac{12 \times (.875)^3}{12} + [10.50 \times (15.3)^2] =$	2,460.7
	Total	= 12,253.8

The radius of gyration is equal to the square root of the quotient obtained by dividing the moment of inertia by the area, that is

$$r = \sqrt{\frac{I}{A}},$$

ATE I.

where $r =$ the radius of gyration of section,

$I =$ moment of inertia of section,

and $A =$ area of section.

In the present case

$$r = \sqrt{\frac{12253.8}{99.05}} = 11.1 \text{ inches.}$$

This is so nearly the value assumed that it will not be necessary to repeat the calculations with the true value of r. If there had been a considerable difference between the assumed and the actual values of r it would have been necessary to repeat the calculations with such changes in the trial section as would furnish satisfactorily correct results.

It must be determined whether the moment of inertia may not be smaller about the vertical axis of symmetry of the section; if this be the case, this smaller value of r must be used to determine the allowed intensity of stress. The calculations for this moment of inertia are as follows:

Cover plate,	$\frac{.625 \times (30)^3}{12} =$	1,405
Upper angles,	$2[4.07 + (12.53)^2 \times 3.9] =$	1,236
Web plates,	$2\left[\frac{(\frac{15}{16})^3 \times 28}{12} + 26.25 \times (11.03)^2\right] =$	6,404
Lower angles,	$2[11.26 + (13.12)^2 \times 4.75] =$	1,656
Bars,	$2\left[\frac{.875 \times (6)^3}{12} + 5.25 \times (13.5)^2\right] =$	1,944
		12,645

This moment of inertia is larger than the one found for the horizontal axis, and it need not therefore be further considered.

The distance from back to back of the lower and upper angles will be made 28¼ inches, being slightly greater than the depth of side plates in order to prevent their interference with the cover plate or lower bars during manufacture. The axis of the pins will be placed 14½ inches from the upper edge of the cover plate, thus allowing a small counter moment from the direct stress due to a lever arm of 0.7 inch. The other upper chord members are treated in precisely the same manner. The stress sheet, Plate 1, shows in sufficient detail the forms of cross-section adopted.

End Post L_0U_1.

The end post is designed under a different provision of the specifications, the allowed intensity of stress for live load being found by the following formula:

$$p = 8{,}500 - 45\frac{l}{r}.$$

The length of the end post between pins is 558 inches, but it may be assumed that the effective length is only one half of this, since the post is supported at its center in one direction by an inclined strut, and in the direction at right angles by the portal bracing. Assuming $l = 279$ inches and $r = 11.1$ inches, the allowed intensity of stress becomes 7,370 pounds per square inch.

Dead load stress $= -371{,}000$ pounds.
Live load stress $= -528{,}000$ pounds.
($\frac{1}{2}$ dead + live) load stress $= 713.500$ pounds.
Therefore $\frac{713{,}500}{7{,}370} = 96.9$ square inches required.

The stress sheet indicates the section adopted for this member. It will be found on examination that the requirements of the specifications concerning the unsupported widths of plates in compression have been fulfilled.

ART. 8. DESIGN OF FLOOR BEAM HANGERS.

U_1L_1, L_4M_4, L_6M_6.

These members are built from shapes rather than from bars with forged ends, in order that the floor beams may be attached to them with the same details that are used at other panel points. The following specifications apply:

§ *30. Floor beam hangers, and other similar members (in tension), liable to sudden loading (plates and shapes) net section at 6,000 pounds per sq. in.*

Long verticals in tension (plates or shapes) net section at 9,000 pounds per sq. in. for live loads and 18,000 pounds per sq. in. for dead loads.

U_1L_1, L_4M_4 and L_6M_6 are long verticals and they are covered by the second of the preceding provisions.

Member U_1L_1.

Dead load stress $= +33{,}000$ pounds.
Live load stress $= +84{,}000$ pounds.
(1/2 dead + live) load stress $= 100{,}500$ pounds.
$\frac{100500}{9000} = 11.17$ sq. ins. required.

The following 15 inch channels spaced 14 inches back to back more than satisfy this requirement:

2, 15″ — [’*s*, weighing 32.95 pounds per lin. ft. @ 9.69 sq. ins. = 19.38 sq. ins. gross or 17.88 sq. ins., net, held together by single latticing 2¼ × ⅜ in.

This is much more metal than is needed, but the member is 36 feet long and requires increased stiffness on that account. As these same channels must be used for a number of the main web-members of the truss, they will also be used for these long verticals.

Members L_4M_4 *and* L_6M_6.

Both of these members are subjected to the same stresses, and since they occupy similar positions in the truss, they will be designed exactly alike.

Dead load stress = + 33,000 pounds.
Live load stress = + 84,000 pounds.
(1/2 dead + live) load stress = 100,500 pounds.
$\frac{100500}{9000} = 11.16$ sq. ins. required.

A narrower channel than 12 inches cannot be used on account of the floor beam connection; it will be necessary to use two 12-inch channels, each weighing 24.51 pounds per lin. ft., with a total gross area of 14.42 sq. ins. and a net area of 12.86 sq. ins. tied together by single latticing 2¼ × ⅜ in. These members will be continued vertically upwards to form the members M_4U_4 and M_6U_6 respectively; their sections are ample to carry the compressive stresses imposed.

ART. 9. DESIGN OF VERTICAL MAIN WEB MEMBERS OR POSTS.

U_2L_2, U_3L_3, U_5L_5.

These main vertical posts are all in compression and therefore subject to the following specification:

§ *33. All posts of through bridges shall be proportioned by the following allowed unit stresses:*

$$P = 8{,}500 - 45\frac{l}{r} \text{ for live load stresses.}$$

$$P = 17{,}000 - 90\frac{l}{r} \text{ for dead load stresses.}$$

Member U_2L_2.

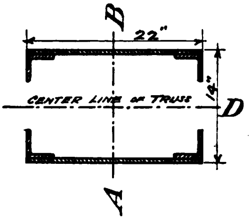
Dead load stress = — 98,000 pounds.
Live load stress = — 161,000 pounds.
(½ dead + live) load stress = — 210,000 pounds.

By assuming the width of post at 22 inches, and the radius of gyration .35 of this width, the length of the post being 540 inches, the allowed intensity of stress becomes

$P = 8{,}500 - \frac{45 \times 540}{7.7} = 5{,}340$ pounds per sq. in., and the area of section required $\frac{210{,}000}{5{,}340} = 39.3$ sq. in.

The following trial section (Fig. 3) may be assumed:

2 plates $22'' \times \frac{9}{16}''$ $= 24.75$ sq. ins.
4 angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{9}{16}'' = \underline{14.60}$ sq. ins.
39.35 sq. ins.



Post U_2-L_2

FIG. 3.

The plates are spaced 14 inches back to back. The moment of inertia about the *AB* axis will be:

$$\text{Plates} = 2\left[\frac{\frac{9}{16} \times (22)^3}{12}\right] = 1{,}000.$$

$$\text{Angles} = 4\,[4.0 + (3.65 \times 10^2)] = \underline{1{,}476}$$

$$\text{Total} = 2{,}476.$$

$$\therefore r = \sqrt{\frac{2{,}476}{39.35}} = 7.95 \text{ ins.}$$

It will not be necessary to make use of the radius of gyration about the *CD* axis (in this case 6.48 inches) since the length of the column to be inserted in the formula, when considering this radius, is much reduced by the cross-bracing at its upper end. The allowed intensity of stress for this seortened column, taken in connection with the smaller radius of gyration, should always be tested.

In the present case, the value of 7.95 inches, for the *AB* radius, checks the value assumed sufficiently near so that recalculation will not be necessary; the trial section will be adopted as it stands using single $2\frac{1}{2} \times \frac{3}{8}$ in. lattice bars.

Member U_3L_3.

Dead load stress = — 32,000 pounds.
Live load stress = + 25,000 pounds.
Live load stress = — 73,000 pounds.

In the case of this member, the following specification relative to alternate stress becomes operative:

§ *35. Members subject to alternate stresses of tension and compression shall be proportioned to resist each kind of stress. Both of the stresses shall, however, be considered as increased by an amount equal to* $\frac{8}{10}$ *of the least of the two stresses for determining the sectional areas by the usual allowed unit stresses.*

The equivalent live load stress for UL_3 then becomes ($\frac{1}{2}$ dead + $\frac{8}{10}$ tension live + compression live) load stress = — 109,000 pounds.

Assuming a width of 15 inches, and the radius of gyration .35 the width, the length of the column being 27 feet (not 54 feet, since it is supported in two directions at the center) furnishes a unit stress of 8,500—(45 × 324)/5.25 = 5,720 pounds per sq. in. The required area will therefore be

$$\frac{109,000}{5,720} = 19.0 \text{ sq. inches.}$$

A trial section of 2–15 in. channels, weighing 32.95 pounds per lin. foot, with a total area of section of 19.38 square inches will be found to furnish a radius of gyration of 5.67 inches, and to fulfill all the required conditions. The spacing back to back of channels will be made uniform with that of the other hangers and posts at 14 inches, and the latticing will be single, 2½ × ⅜ inches

Member U_6L_6.

Dead load stress = — 7,000 pounds.
Live load stress = + 27,000 pounds.
Live load stress = — 79,000 pounds.

The equivalent live load stress is, ($\frac{1}{2}$ × 7,000 + $\frac{8}{10}$ × 27,000 + 79,000) = 104,100 pounds compression.

The effective length of column may be assumed as one half of 63 feet, and since the stress in U_6L_6 is nearly the same as that in U_3L_3, the section belonging to the latter may be used for a trial; therefore $r = 5.67$ inches. The allowed intensity of stress then becomes: $p = 8,500 - (45 \times 378)/5.67 = 5,500$ pounds per sq. in., and the area of cross-section required 104,100/5,500 = 18.95 sq. ins.

The trial section of 19.38 sq. ins. may therefore be adopted as final, using as in the other cases a spacing of 14 inches back to back of channels, and single latticing 2½ inches by × ⅜ inch.

Tension Web Members, U_2L, U_3M_4, M_4L_5, U_5M_6.

The following specification applies to these tension web members:

§ *30. The allowed unit stresses in tension for main diagonals and counters shall be:*

10,000 pounds per square inch for live loads.
20,000 pounds per square inch for dead loads.

Member U_2L_3

Dead load stress = + 78,000 pounds.
Live load stress = + 160,000 pounds.

The equivalent live load stress is (½ dead + live) load stress = + 199,000 pounds.

The required order of cross-section will therefore be

$$\frac{199{,}000}{10{,}000} = 19.9 \text{ sq. ins.}$$

and will be formed of 2 eye-bars, 7 inches × $1\frac{7}{16}$ inches = 20.12 sq. ins.

The following table shows the method of design for the other web members.

Member.	Dead Load Stress in Pounds	Live Load Stress in Pounds.	Equivalent Live Load Stress in Pounds.	Area Required Square Inches.	Section Adopted.	Area of Final Section Square Inches.
U_3M_4	+90,000	+204,000	+249,000	24.9	2 bars — 7″ × $1\frac{13}{16}$″	25.38
M_4L_5	+57,000	+168,000	+196,500	19.65	2 bars — 7″ × $1\frac{7}{16}$″	20.12
U_5M_6	+36,000	+173,000	+191,000	19.1	2 bars — 7″ × $1\frac{3}{8}$″	19.26

ART. 10. DESIGN OF COUNTER MEMBERS, L_2U_3, L_3M_4, M_4U_5 AND L_5M_6.

The following specification applies to these members:

§ *45. The areas of counters shall be determined by taking the difference in areas due to the live and dead load stresses considered separately; the counters in any one panel must have a combined sectional area of at least three square inches, or else must be capable of carrying all the counter live load in that panel.*

Member L_2U_3.

There is actually no counter stress in the panel L_2L_3, and the member L_2U_3 is only inserted to secure a margin of safety. In compliance with the specifications, therefore, it must have a cross-section of at least 3 square inches. One adjustable rod 1¾ in. square, having an area of 3.06 square inches fulfills these requirements.

Member L_3M_4.

Dead load stress = — 90,000 pounds.
Live load stress = + 82,000 pounds.

The difference between the equivalent of the dead load stress in terms of the live load, and the live load will be — ½(90,000) + 82,000 = 37,000 pounds tension.

At 10,000 pounds per square inch, a cross-sectional area of 3.7 square inches will be required. Two adjustable rods 1½ inches square, having an area of cross-section of 4.5 square inches will be used.

The stress in the remaining counter members are not completely determined statically, and the sectional areas of those members should be based on the undiminished live load stresses in them.

Member L_5M_6.

Dead load stress = — 36,000 pounds.
Live load stress = + 136,000 pounds.

This member should be designed according to the last condition of the specification; viz., to be capable of carrying all the counter live load in the panel; the area required will therefore be 13.5 square inches, and will be composed of 2 adjustable bars 6 × 1⅛ ins., having an area of 13.5 square inches.

Member M_4U_5.

Live load stress = + 39,000 pounds,
Dead load stress = + 62,000 pounds,

or

Dead load stress = + 104,000 pounds,
Live load stress = — 66,000 pounds.

This member will be designed according to the same condition as L_5M_6. In this case all the counter live load stress equals 104,000 pounds, and at 10,000 pounds per square inch, the area required would be 10.4 square inches, which is supplied by two 6 × 1 in eye-bars having an area of 12 square inches.

This completes the design of the main members of the truss;

the intermediate struts M_3M_4, M_5M_6 and the collision strut supporting the center of the end post carry no definite stresses. Their form of section depends to a great extent upon their end connections, and they are usually built of light angles latticed together in box shape.

In the present case all these struts will be built of four $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ in. angles, having a combined area of 10 square inches laced together in a box form by four sets of single lacings $2\frac{1}{4} \times \frac{5}{16}$ in. The dimensions of the box forms will depend upon the connections which these struts make with other members.

ART. 11. COMBINED STRESSES.

§ *39. When any member is subjected to the action of both axial and bending stresses, as in the case of end posts of through bridges, or of chords carrying distributed floor loads, it must be proportioned so that the greatest fiber strength will not exceed the allowable limits of tension or compression on that member. If the fiber stress resulting from weight only, of any member exceeds 10 per cent. of the allowed unit stress on such member, such excess must be considered in proportioning the areas.*

In accordance with the above specification, the chord members should be tested with reference to the bending stresses induced by their own weight. The condition of the ends or of the supported sections of these sections considered as beams is largely a matter of conjecture; they may be considered fixed at their ends, or assumed entirely free to turn, or their condition may be assumed half way between. It is probably reasonable, at least, to assume these members fixed at their ends or supported sections, since the friction on the pins will tend to produce that condition.

It will be sufficient for present purposes to test one upper and one lower chord member for combined stresses. The method here adopted for finding the bending stress in the truss members, although always yielding safe results, is not exact. Strictly speaking, the bending effect of the axial stresses should be recognized.*

Member U_5U_6.

The cross-sectional area of this number is 99.09 square inches but allowing 20 per cent. additional for rivets, latticing and other details, the weight per inch of length will be 33.8 pounds. The length between centers of end pins is 350 inches; therefore the maximum bending moment M, assuming the member to have fixed ends, will be at the center, and equal to 518,000 inch pounds.

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* For exact theory, see Burr's "Resist. of Mat.," edition 1903, pp. 181.

The moment of inertia I of the section is 12,254; therefore k, the intensity of stress in the extreme fibers is, by the following formula:

$$k = \frac{Md}{I} = \frac{518{,}000 \times 14}{12{,}254} = 592 \text{ pounds per square inch.}$$

This is less than 10 per cent. of the allowed compressive intensity in the member, and it may therefore be neglected.

Member L_5L_6.

The dimensions of one eye-bar are 8 in. deep × 1⅞ in. thick × 350 in. long; the weight per inch of length will be 4¼ pounds, and the maximum bending moment at the center, assuming fixed ends, will be 32,500 inch pounds. The amount of inertia is 80; therefore the stress in the extreme fibers at the center will be

$$\frac{32{,}500 \times 4}{80} = 1{,}625 \text{ pounds per squre inch.}$$

The permissible axial unit stress (see page 263) was 15,750 pounds per square inch, and the bending stress is so slightly in excess of 10 per cent. that it may be safely neglected.

There still remains to be treated the bending in the end post caused by the overturning effect of the wind; the maximum bending will occur at the connection of the curved bracket of the portal to the end post and at the foot of the end post, since that member may be considered fixed at the lower end, and also fixed in direction from the extreme upper end to the bracket connection. A point of contraflexure midway between the foot of the end post and the curved bracket may therefore be assumed to exist.

If it be supposed that the wind loads of the upper chord are carried equally to both end posts, the maximum bending moment becomes ½ wind reaction by ½ the distance along the post measured from its foot to the bracket connection; in this case 14,875 pounds × 140 inches = 2,082,500 inch pounds. The moment of inertia about the axis of bending is about 12,500, and the distance from the neutral axis to the extreme fiber of the upper angles is 15 inches, it being remembered that the bending takes place in the plane at right angles to the vertical plane through the axis of the truss. The stress in the extreme fiber will then be

$$\frac{2{,}082{,}500 \times 15}{12{,}500} = 2{,}500 \text{ pounds per square inch.}$$

At the same time the stress in the web plates, which constitute the greater portion of the cross-section of the post, will be $(2{,}082{,}500 \times 11.5)/12{,}500 = 1{,}920$ pounds per square inch, due to this same bending.

This stress is a wind stress, and is therefore subject to § 36 of the specifications. The average stress in the end post due to dead and live loads is $(371{,}000 + 528{,}000)/97.25 = 9{,}250$ pounds, and one quarter of this is 2,310 pounds; therefore the bending due to wind may be safely neglected. As a further margin of safety it should be noted that the point of greatest bending due to wind does not coincide with the point of greatest bending due to the simple compression in the member.

ART. 12. DESIGN OF STRINGERS AND FLOOR BEAMS.

The following specifications apply to these members:

§ *30. The allowed unit stress in tension, for the bottom flanges or riveted cross girders (net section), shall be 10,000 pounds per square inch, and for the bottom flanges of riveted longitudinal plate girders used as track stringers (net section), 10,000 pounds per square inch.*

§ *40. In beams and plate girders the compression flanges shall be made of the same gross section as the tension flanges.*

§ *41. Riveted longitudinal girders shall have, preferably, a depth not less than of one tenth the span.*

§ *42. Plate girders shall be proportioned upon the supposition that the bending or chord stresses are resisted entirely by the upper and lower flanges, and that the shearing or web stresses are resisted entirely by the web plate; no part of the web plate shall be estimated as flange area.*

The distance between centers of gravity of the flange area will be considered as the effective depth of all girders.

§ *43. The webs of plate girders must be stiffened at intervals of about the depth of the girders, wherever the shearing stress per square inch exceeds the stress allowed by the following formula:*

Allowed shearing stress =

$$\frac{12{,}000}{1 + \frac{H^2}{3{,}000}}$$

where H = ratio of depth of web to its thickness; but no web plates shall be less than 3/8 of an inch in thickness.

The following specifications for riveting also apply to the design of these members:

§ *37. The rivets in all members, other than those of the floor and lateral systems, must be so spaced that the shearing stress per square inch shall not exceed 9,000 pounds; nor the pressure on the bearing surface (diameter × thickness of the piece) of the rivet hole exceed 15,000 pounds per square inch.*

The rivets in all members of the floor system, including all hanger connections, must be so spaced that the shearing stresses and bearing pressures shall not exceed 80 per cent. of the above limits.

In the case of field riveting (and for bolts) the above-allowed shearing stresses and pressures shall be reduced one third.

Rivets and bolts must not be used in direct tension.

The stringers are assumed 350 inches long and 32¼ inches back to back of angles, the depth of the stringer being taken a little less than one tenth of the span to permit the required track elevation. The actual depth of the web plate will be 32 inches, but the distance back to back of angles is increased by ¼ inch, so that the web will not project between the angles, and cause difficulty in riveting the cover plates. It should be noted that shallow stringers deflect more than deeper ones, thus tending to produce greater direct tensile stresses on the end connection rivets.

The maximum bending moment due to live load will be found for that position of loading in which the center of gravity of the load is situated as far on one side of the center of the beam as the point of maximum bending is on the other side, and a wheel load will always be found over this point of maximum bending. In the present case, this point is found 14.2 ft. from the left end of the stringer under the second driver of the locomotive, as shown by Fig. 4.

The reaction at the left end of the stringer for this position of the loading is 43,800 pounds; the maximum bending moment is then found as follows:

$$M = 43{,}800 \times 14.2 - 10{,}000 \times 13 - 20{,}000 \times 5 = 392{,}000 \text{ ft. lbs.}$$
$$= 4{,}704{,}000 \text{ in. lbs.}$$

The dead load bending moment at the same point may be taken equal to the maximum dead load bending moment at the center, and is found as follows:

The track is assumed to weigh 400 pounds per lin. ft. and both stringers, and bracing 375 pounds per lin. ft. The weight per lin. inch per stringer will then be 32.29 pounds and the maximum moment:

$$\frac{32.29 \times (350)^2}{8} = 494{,}000 \text{ in. lbs.}$$

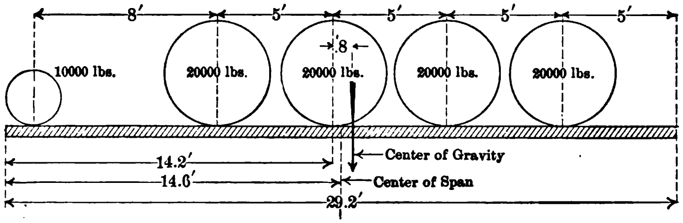

FIG. 4.

The effective depth of the girder according to the specifications may be assumed at 30 inches, and since the allowed intensity of stress is 10,000 pounds per sq. in., the required net section will be:

$$\frac{4{,}704{,}000 + 494{,}000}{30 \times 10{,}000} = 17.3 \text{ sq. ins.}$$

This flange or chord area will be afforded by 2 angles $6 \times 6 \times \frac{9}{16}''$ having a gross section of 13 sq. ins. = 10.7 sq. ins. net, and 1 cover plate $14 \times {}^{9}\,''$ having a gross section of 7.9 sq. ins.= 6.8 sq. ins. net. Total, 17.5 sq. ins. net.

Length of Cover Plate.

The length of the cover plate will be determined by assuming that the bending moment follows a parabolic law. If L represents the total length of the stringer, l the length of the cover plate, A the total section of the flange, and a the total area of all the cover plates counting from the outside and including the area of the one under consideration, then

$$l = L\sqrt{\frac{a}{A}}.$$

Inserting the requisite quantities in the formula,

$$l = 29.2\sqrt{\frac{6.8}{17.3}} = 18.3 \text{ feet.}$$

At least six inches should be added to each end of the completed length of a cover-plate in which to place additional rivets, so that the plate may actually develop its full strength at the section where it is needed. The length of the cover-plate in this case will therefore be 19.3 feet.

Web Plate.

The maximum shear for both dead and live loads occurs at the end of the stringer. The live load shear with the first driver at this point is 62,500 pounds. The dead load shear is 5,700 pounds, thus making the total shear 68,200 pounds. The working shear in the web plate may be taken at 5,000 pounds per square inch of gross section. The required cross-section will then be 13.6 square inches, and the web plate will be taken $32 \times \frac{7}{16}''$ = 14.0 sq. ins. According to the specifications, if the shearing stress exceeds $12{,}000/[1 + (H^2/3{,}000)]$, intermediate stiffeners will be required. In the present case this formula has a value of $12{,}000/1 + [(73.2^2/3{,}000)] = 4{,}310$ pounds per square inch, whereas the actual intensity of shear in the stringer is 5,000 lbs. per square inch at the ends, with a constantly decreasing value towards the center of the stringer; no intermediate stiffeners will therefore be required. The end stiffeners cannot be subjected to a rational analysis; in the present case they will be composed of two $6'' \times 6'' \times \frac{7}{16}''$ angles, being chosen of such dimensions as to permit of making simple connections to the floor beams. They are sometimes designed by requiring that their normal section must carry the end reaction with a certain specified unit process (as 7,000 pounds per square inch). In this case the normal section is 10.12 inches, and it is ample for the purpose.

Riveting.

The following rule will determine the pitch in the flanges of a girder; viz., the pitch of the riveting at any section of the flanges will be the quotient obtained by dividing the product of the vertical distance between the rows of riveting in the two flanges and the allowed stress in one rivet either for shear or bearing, by the maximum shear at the section. This rule is readily demonstrated by taking the difference between the bending moments at two adjacent rivets.

In the present case the pitch at the ends of the stringer will be

$$\frac{30 \times (5{,}740 \times 80\%)}{68{,}200} = 2''.1.$$

At 10 feet from the ends the dead load shear will be 1,925 pounds, and the maximum live load shear, with the first driver over the section, will be 31,300 pounds.

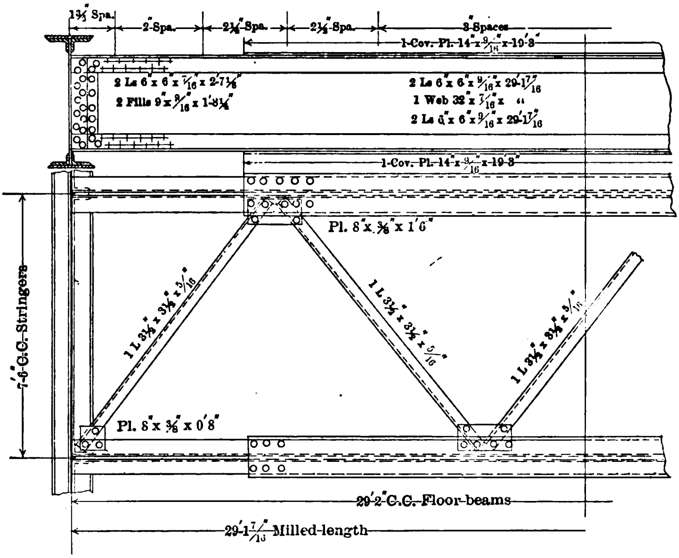
1¾" Spa.
2" Spa.
2⅛" Spa.
2⅛" Spa.
3" Spaces
1-Cov. Pl. 14" x 9/16" x 19' 3"
2 Ls 6" x 6" x 7/16" x 2' 7⅛"
2 Fills 9" x 9/16" x 1' 8¼"
2 Ls 6" x 6" x 9/16" x 29' 1 7/16"
1 Web 32" x 7/16" x "
2 Ls 6" x 6" x 9/16" x 29' 1 7/16"
1-Cov. Pl. 14" x 9/16" x 19' 3"
Pl. 8" x ⅜" x 1' 6"
1 L 3½" x 3½" x 5/16"
1 L 3½" x 3½" x 5/16"
1 L 3½" x 3½" x 5/16"
7' 6" C.C. Stringers
Pl. 8" x ⅜" x 0' 8"
29' 2" C.C. Floor-beams
29' 1 7/16" Milled-length

FIG. 5.

The pitch at this point will therefore be

$$\frac{30 \times (5{,}740 \times 80\%)}{31{,}300 + 1{,}925} = 4''.15.$$

The flange rivets have thus far been considered to carry the direct horizontal stress only, but those in the upper flange must also carry as a vertical load the weight of the driving wheels as they pass. One driver, weighing 20,000 pounds, may be supposed to be distributed over three ties, or over a distance of 3 feet, assuming 6-inch spaces between ties 8 inches wide. The greatest effect in increasing the pitch will be found at the center of the girder. At this point 9 rivets would approximately be required in a distance of 3 feet, if the longitudinal stress alone existed; and 20,000/4,592 = 4.4 rivets for the vertical load alone. The proper number of rivets for this distance of 3 feet is then $\sqrt{9^2 + 4.4^2} = 10$. It is seen that the effect of this vertical loading in increasing the number of rivets is very small. The spacing finally adopted is shown in Fig. 5; it varies from 1¾ inches at the ends to 3 inches at the center, and more than provides for vertical load effect.

Rivets in Cover Plates.

The stress carried by the cover plate will be 6.8 square inches × 10,000 pounds = 68,000 pounds. The permissible stress in the rivets will be determined for single shear, at 80 per cent. of 5,410 pounds = 4,328 pounds per rivet. Sixteen rivets will be required. They will be placed in four rows with 3 inches pitch for a distance of 15 inches from the ends of the plate, and 6 inches pitch for the remaining distance.

Rivets on End Connection.

The rivets fastening the stringers to the floor beam are field rivets, and therefore the allowed intensities of stress in them must be reduced one third.

Two separate conditions effect the number of rivets at this end connection; first, the single shear on these rivets when one stringer has its maximum reaction, and secondly, the bearing on the web of the floor beam, when the combined reactions at the two stringer ends resting on the floor beam have their maximum value. For the first case, each rivet in a single shear will carry 5,410 × .8 × ⅔ = 2,885, and with a maximum end reaction of 68,200 pounds, there will be required 24 rivets; in the second case, assuming the thickness of the floor beam web plate at $\frac{9}{16}$ inch, the bearing capacity of one rivet will be 7,380 × .8 × ⅔ = 3,930 pounds; the simultaneous maximum combined reactions of the two stringers will be, for dead load 14,000 pounds, and for live load 84,000 pounds (see design of floor beam for these reactions). or a total of 98,000 pounds; there will therefore be required 25 rivets. Thirteen rivets will however actually be placed in each 6 × 6 inch angle, making 26 rivets at this connection.

The bill of material with the weights for one stringer, and $\frac{1}{2}$ the cross bracing, using that indicated in Fig. 5 for which no actual design will be made, may be written as follows:

			Lbs. per Ft.	
1 web plate $32 \times \frac{7}{16}''$	29.1	ft. long	@ 50.58 =	1,470 lbs.
4 angles $6 \times 6 \times \frac{9}{16}''$	"	" "	@ 21.9 =	2,550 "
2 cover plates $14 \times \frac{9}{16}''$	19.25	" "	@ 26.78 =	1,060 "
4 end stiffeners $6 \times 6 \times \frac{7}{16}''$	2.6	" "	@ 17.2 =	178 "
4 filling plates $6 \times \frac{9}{16}''$	1.6	" "	@ 11.48 =	74 "
$2\frac{1}{2}$ bracing angles $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}''$	—9.5	" "	@ 2.1 =	50 "
About 450 pairs of $\frac{7}{8}''$ rivet heads			@ 22.2 lbs. per 100 =	200 "
				5,582 lbs.

The actual weight per foot of stringer is thus seen to be 384 pounds, or 9 pounds per foot more than was assumed. This difference is so small as to require no correction in the design, but it is better to have the actual weight under rather than over the assumed.

Intermediate Floor Beams.

The maximum live load floor beam reaction occurs when wheel 5 is placed directly over the floor beam. Wheels 1 to 4 will then be on one adjoining stringer and wheels 5 to 9 on the other, making the resulting reaction 84,000 pounds.

Assuming the dead weight of the floor beam to be 300 pounds per lin. ft., the dead load reaction will become:

Dead weight of one stringer = 11,300 pounds.
Dead weight of $\frac{1}{2}$ floor beam = 2,700 "
14,000 pounds.

The sum of the dead and live load reactions will then be 98,000 pounds.

The stringers are spaced 7 feet 6 inches apart, and since the distance between trusses is 18 feet, the stringer is distant 63 inches from the center line of the truss. Therefore the maximum bending of the floor beam will be $98{,}000 \times 63 = 6{,}170{,}000$ in. lbs. Assuming an effective depth of 42 inches, the flange area required will be $6{,}170{,}000/42 \times 10{,}000 = 14.69$ sq. ins. This section will be composed of 2 angles $6'' \times 4'' \times \frac{1}{2}'' = 9.6$ sq. ins. gross = 8.6 sq. ins. net; 1 cover plate $14'' \times \frac{1}{2}'' = 7.0$ sq. ins. gross = 6.0 sq. ins. net Total, 14.6 sq. ins. net.

The cover plate should have the following length:

$$l = 18\sqrt{\frac{6}{14.6}} = 11.5 \text{ feet;}$$

owing to the details at the ends of the floor beam the upper cover plate will have a length of 12 feet 7 inches and the lower a length of 14 feet 9 inches.

On account of the large number of rivets made necessary at the end connection of the floor beam, and the consequent weakening of the plate, the assumed intensity of shearing stress for the web has been taken at 4,000 pounds per square inch of gross section. The area required will therefore be $98{,}000/4{,}000 = 24.5$ square inches, and a plate 43 by $\frac{9}{16}$ inch = 24.2 square inches will fulfill the requirements. No intermediate stiffeners will be required, as the end connections of the stringers render end stiffeners unnecessary. The distance from back to back of the flange angles will be $43\frac{1}{4}$ inches. At the connection of the floor beam to the post, a considerable portion of the web plate must be removed in order to leave room for the heads of the lower chord eye-bars. It is therefore customary to cut the web plate at some distance from the end, and to splice to it another plate of irregular shape having an equal cross-sectional area. The general detail drawing, Plate II., illustrates the method adopted in the present case. The central portion of the web is 10 feet 12 inches long and spliced to it at each end by means of two $\frac{7}{16}$-inch plates, placed over $\frac{1}{2}$-inch fillers, is another plate designated on the figure as "web 24 inches × $\frac{9}{16}$ inches × 6 feet 3 inches long." To this end web plate are fastened the end stiffeners formed of two $6 \times 3\frac{1}{2} \times \frac{7}{16}$ inch angles. The end stiffener must be divided into an upper and lower part in order to allow the upper flange of the floor beam to abut against the post. The lower portion of the end web plate will be cut away with rounded edges, to the form required by the disposition of the eye bars, all floor beams being made alike to conform to that post in which the greatest clearance is required. This edge will be protected by two angles $3\frac{1}{2}$ inches × $3\frac{1}{2}$ inches × $\frac{3}{8}$ inch, and 3 feet 6 inches long. The lower flange of the floor beam will be connected to the foot of the post by a flat plate riveted to both, to provide against swaying, and to form part of the lower lateral wind system.

Riveting.

Instead of employing the rule used in finding the pitch of the rivets in the flanges of the stringers it will be better to divide the amount of stress in the flanges of these beams between the post connection and the stringer connection, by the allowed stress capacity of one rivet; the quotient will determine the number of rivets between these two points. For example:

$$\frac{146,900}{7,380 \times .8} = 25 \text{ rivets.}$$

These are easily provided for in the lower flange, but a pair of supplementary angles $6 \times 3\frac{1}{2} \times \frac{7}{16}$ inch and 3 feet $1\frac{1}{2}$ inches long must be fastened to the upper flange at its ends in order to provide for the requisite rivets to transfer the stress from the web plate to the flanges. The rivet design of this end connection is not very precise. The rivets piercing the spliced central portion of the web plate of the floor beam really aid in carrying the stress through the splice plate to the flanges. It is essential that all parts at this point shall be firmly and rigidly tied together. The pitch in the central portion of the beam will be 6 inches.

The number of rivets necessary on each side of the splice will be

$$\frac{98,000}{7,380 \times .8} = 17,$$

and they will be placed in two rows, as shown in Plate II.

At least the same number of rivets will be necessary to fasten the end stiffeners to the web plate; but since 17 rivets cannot be placed in one row, the two rows shown on the plate will be inserted, even though all those there shown are not necessary. The resulting surplus may be considered as compensating for the tendency to pull on the rivet-heads, due to the deflection of the loaded beam.

The end stiffeners are connected to the post by field rivets. The thickness of the web of the post is three eighth inch, but this thickness will be increased by a diaphragm which will later be inserted between the channel webs, so that the single shear of the rivets determines their number.

The rivets required will therefore be $[98,000/(5,410 \times .8 \times \frac{3}{8})] = 34$.

The rivets in the cover plate will be pitched 3 inches for a distance of 15 inches from the ends of the cover plate, and 6 inches for the remaining distance.

A small angle bracket consisting of a $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}''$ angle 1 foot $0\frac{1}{2}$ inches long will be riveted to the floor beam to form a seat for the ends of the stringer during erection. This bracket is used for temporary convenience only, and is not supposed to carry any stress after erection.

It is sometimes customary, if web plates have no splices, to take one sixth [or sometimes one eighth] of the web section as acting in either flange. If no rivet holes were punched for the stiffeners, this method would be allowable. But such rivet holes frequently take out considerable metal, and as the tension side of the plate only is affected, one sixth of the remaining metal ceases to be a proper proportion. On the whole, therefore, it is better to neglect the bending resistance of the web, and allow it to balance, so far as it may, the effect of the rivet holes being out of the center of gravity of the flange angles.

The bill of material and the weight of the floor beam is given in the following table:

	Length.	Weight per Foot	Total Weight.
1 Web plate 43″ x 9/16″	12′ 10″	82.24	1040
2 " plates 24″ x 9/16″	6′ 3″	45.90	573
2 Flange angles 6″ x 4″ x ½″	16′ 10″	16.2	545
2 " " "	14′ 9″	16.2	478
1 Cover plate 14″ x ½″	12′ 7″	23.8	300
1 " " "	14′ 9″	23.8	350
4 End stiffening angles 6″ x 3½″ x 7/16″	1′ 8¾″	13.5	90
4 " " " "	2′ 7″	13.5	140
4 " fillers 6″ x 7/16″	2′ 4″	8.93	83
4 " finishing angles 3½″ x 3½″ x ⅜″	3′ 6″	8.5	119
4 Splice plates 24″ x 7/16″	3′ 6″	35.70	500
4 Filling plates 30″ x ½″	3′ 3″¼	51.00	660
4 Upper flange, end angles 6″x3½″x7/16″	3′ 1½″	13.5	163
4 " " fillers 6″ x ½″	2′ 1½″	10.20	86
2 " " " 3½″ x 9/16″	1′ 1½″	6.70	15
4 Bracket angles 3½″ x 3½″ x ⅜″	1′ 0½″	8.5	35
340 prs. of rivet heads at 22.2 lbs. per 100			150
			5,327 lbs.

The assumed weight is 5,400 pounds *i. e.*, greater than the actual, as it should be.

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THE DETAILED DESIGN OF A RAILROAD BRIDGE.
PART II.

BY W. H. BURR, C.E., AND M. S. FALK, C.E.

ART. 13. SPECIFICATONS FOR LATERAL AND WIND BRACING.

The following specifications apply to the bracing:

§ *30. Allowed unit stress, in tension, for longitudinal, lateral and sway bracing for wind and live load stresses = 18,000 pounds per sq. in.*

§ *31. Angles subject to direct tension must be connected by both legs, or the section of one leg only will be considered as effective.*

§ *33. Allowed unit stress, in compression, for lateral struts and rigid bracing* $= P = 13{,}000 - 70\, l/r$ *for wind stresses, where* P, l, *and* r *have the same meaning as previously. No compression member, however, shall have a length exceeding 125 times its least radius of gyration. Lateral struts, with adjustable bracing, will be proportioned by the above formula to resist the maximum due either to the wind and load or to an assumed initial stress of 10,000 pounds per sq. in. on all the rods attached to them.*

§ *35. Members subjects to alternate stresses of tension and compression shall be proportioned to resist each kind of stress. Both of the stresses shall however be considered as increased by an amount equal to eight tenths of the least of the two stresses, for determining the sectional area by the above allowed unit stresses.*

§ *37. The rivets in the lateral and sway bracing will be allowed 50 per cent. increase over usual allowed unit stresses.*

§ 94. *Where rods are used in the lateral, longitudinal or sway bracing, they shall be square bars, but in no case shall they have a less area than one square inch. Rods with bent eyes must not be used.*

§ 95. *All through bridges shall have latticed portals, of approved design, at each end of the span, connected rigidly to the end posts and top chords. They shall be as deep as the specified head room will allow.*

§ 96. *Where the height of the trusses exceeds 25 feet, an approved system of overhead diagonal bracings shall be attached to each post, and to the top lateral struts.*

§ 97. *All bars and rods in the web, lateral, longitudinal or sway systems, must be securely clamped at their intersections to prevent sagging and rattling.*

ART. 14. DESIGN OF UPPER LONGITUDINAL WIND BRACING.

The tension members will be formed of angles connected by both legs, so that the full area will become available. The following table shows clearly the method of design, the allowable unit stress being 18,000 pounds per sq. in.:

Member.	Stress in Pounds	Area Required sq ins.	Section Adopted.	Gross Area of Section.	Net Area of sec. in sq. ins.	Rivets Required at End.
U_1U_2	+42,500	2.36	1–3½ x 3½ x $\frac{7}{16}$″ angle.	2.88	2.50	6
U_2U_3	+33,000	1.84	1– “ “ $\frac{3}{8}$ “	2.50	2.17	5
U_3U_4	+23,700	1.33	1 “ “ “ “	2.50	2.17	4
U_4U_5	+14,300	0.80	1– “ “ $\frac{5}{16}$ “	2.09	1.81	2
U_5U_6	+ 4,700	0.28	1– “ “ “ “	2.09	1 81	1

These members are connected to the upper chord by $\frac{3}{8}$-inch gusset plates. The bearing capacity of one machine-driven rivet in this thickness of plate will therefore be 4,920 × 150 per cent. = 7,380 pounds, and this quantity determines the number of rivets required at the end connections as given in column 7, although all the connections will be made uniform with six rivets. The compression members at right angles to the axis of the bridge are not designed by any exact theory. They will be formed either of cross frames or struts, as indicated on the stress sheet, and they are more than ample in section to sustain the direct compressive stresses there indicated; in fact, since their main duty is to stiffen the structure and to cause the two trusses to act as a whole, they are made as rigid as the judgment of the engineer requires. The portal bracing is designed in a somewhat similar way, although tested by computations in its various parts.

The struts will be placed at those points, U_4 and U where the upper chord is continued in a straight line, and where there are no inclined web members. They will be composed of four angles, each $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$ inch, built up in I form with double lattice bars $2\frac{1}{4} \times \frac{3}{8}$-inch, and connected to two gusset plates, one lying on top of the cover plate of the upper chord and the other fastened to the bottom angle of the upper chord.

The cross frames will be placed at all the other panel points. They will be formed of an upper strut, like that just described, and a lower strut placed at the upper clearance limit and formed of four angles each $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$ inch, latticed together in I form by bars $2\frac{1}{4} \times \frac{5}{16}$ inch. The lower strut will be fastened to the post by means of a $\frac{5}{16}$-inch gusset plate lying between two $3\frac{1}{2} \times 3 \times \frac{5}{16}$ inch angles, two feet long, the latter being riveted to the center of the post by 12 rivets. Between the two I struts will then be placed a compound lattice work composed of single $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$-inch angles tied together at intersections by small square plates with five rivets at each crossing and fastened to the struts by $\frac{5}{16}$-inch gusset plates, the details of which are shown in Plate II. The cross frames are really the intermediate transverse bracing, the theory of whose design would be determinate if it were not the duty of the upper horizontal lateral bracing to transfer all the wind load to the upper ends of the end posts. It is possible to design the cross frames by ignoring this upper wind bracing. Their main function however, is to stiffen the bridge under rapidly moving train loads, rather than to give additional stability against wind loads.

ART. 15. DESIGN OF PORTAL BRACING.

The portal bracing will be made as rigid as possible and will take the form of a latticed girder placed in the plane of the pins of the end posts, as shown in outline on the stress sheet. The upper and lower flanges will be made alike of two $6 \times 4 \times \frac{3}{8}$-inch angles with a $14 \times \frac{3}{8}$-inch web plate. The two flanges will be tied together by a compound latticing composed of single $5 \times 3\frac{1}{2} \times \frac{3}{8}$-inch angles. The upper flange will be fastened to the cover plate of the top chord, U_1U_2, by means of a bent gusset plate, as shown in Plate II.; the lower flange will be fastened to the side plate of the end post in a manner similar to that of the cross frames. The lower flange will, moreover, be strengthened at each end connection by means of a plate and angle bracket, composed of a $\frac{3}{8}$-inch plate and $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$-inch angles, as shown in Plate II. The outer end of the bracket will be supported from the latticing of the portal frame by a single $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$-inch angle.

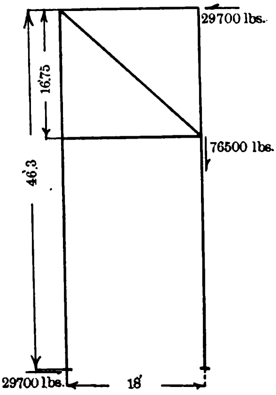

FIG. 6.

The wind force acting at the upper end of the end post is 29,700 pounds. The length of the end post is 46.3 feet, and the distance between trusses 18 feet. The increase of load on one truss, which acts as if hung at the connection point of the lower flange of the portal, will therefore be 76,500 pounds. The multiple bracing employed produces ambiguity in the stresses of the portal members. If the assumption be made, as shown in the accompanying figure, that the multiple bracing is replaced by a single tension-bar, and that the wind reaction is entirely resisted at the foot of the leeward post, the stress in the lower flange will be: $\frac{76,500 \times 18}{16.75} = 82,200$ pounds compression. The allowed intensity of stress is $P = 13,000 - 70\ l/r$, in which $l = 216$ inches and r may be taken approximately as 3 inches. Then $P = 8,000$ pounds per sq. in., approximately, and the sectional area required will be $\frac{82,200}{8,000} = 10.4$ sq. ins.

The actual area will be:

2	$6 \times 4 \times \frac{3}{8}$-inch angles	= 7.22 sq. ins.
1	$14 \times \frac{3}{8}$-inch plate,	= 5.25 " "
		12.47 sq. ins.

The section adopted is therefore sufficient.

The stress in the upper flange may be taken to be equal to the wind reaction transmitted from one truss to the other; its section, although so taken, need not be as great as that of the lower flange.

ART. 16. DESIGN OF LOWER LONGITUDINAL WIND BRACING.

Although the inclined web members in this horizontal truss may suffer apparently reversal of stress, it must be remembered that the two systems of bracing provide for opposite directions of the wind without such reversal. The specifications in regard to reversal of stresses do not therefore become operative. If it be assumed that the inclined web members act in compression, they must be designed with the prescribed unit compressive stresses. As they are fastened at two points to the stringers and at the center to each other, it may be proper also to assume the effective length of a member to be the distance from its end to the point of intersection with the center line of the stringer, *i. e.*, in this case about 120 inches. These members will be formed of two unequal legged angles, with the longer legs vertical and riveted back to back. The following table shows the sections adopted:

Member.	Stress in Pounds. Compression.	Radius of Gyration in ins. Final Section.	Allowed Unit Stress in lbs per sq. in.	Area Required in sq. ins.	Section Adopted. 2 Angles	Actual Section in sq. ins.	Number of Rivets Required at End Connection.
L_0L_1	71,600	1.56	7610	9.4	5x4x$\frac{9}{16}$"	9.52	14
L_1L_2	59,200	1.56	7610	7.8	" " ½	8.58	11
L_2L_8	47,700	1.56	7610	6.3	" " ⅜	6.46	10
L_3L_4	36,900	1.56	7610	4.85	" " "	6.46	7
L_4L_5	26,800	1.23	6170	4.35	4x4x⅜	5.7	6
L_5L_6	17,500	1.23	6170	2.84	" " "	5.7	4

It will be seen that the sections adopted for the panels near the center of span are greater than actually needed; but since a portion of the duty of these members is to stiffen the structure under rapidly moving loads their section may properly be greater than required by the computed stresses.

The connection of these members to the posts is made by a flat plate, ½ inch thick, which in turn is fastened to the foot of the post by 3½ × 3½ × ⅜-inch angles, as shown in Plate II. The amount of stress which these angles must carry to the plate is the difference of wind stresses found in the two adjacent lower chord panels which has a maximum value of 222,000 — 122,000 = 100,000 pounds at the foot of U_1L_1.

The specifications for wind bracing riveting show the single shearing capacity of one rivet to be 5,410 × 150 per cent. × ⅔ = 5,410 pounds, and the bearing capacity in a ⅜-inch thickness of metal to be 4,920 × 150 per cent. × ⅔ = 4,920 pounds. At the foot of U_1L_1 there will therefore be required $\frac{100,000}{4920} = 21$ rivets; since the two angles inside the post do not afford space for all these rivets, a U-shaped plate will be riveted within the post, which will permit placing the required number of rivets. At the foot of U_2L_2, the maximum stresses carried by the plate will be 78,000 pounds, and will require 16 rivets. At the foot of U_3L_3, the stress will be 56,000 pounds, and 12 rivets are required. Plate II. shows the disposition of these rivets in the angles. The number of rivets connecting the ends of the lateral bracing to the plate, as given in column 8 of the above table, was found in the same way as for the upper chord.

The lower flanges of the floor beams act as struts in the lower lateral system. The ½-inch plate which is used as a connection at the foot of the posts, is sufficiently extended to be firmly riveted to the lower flange of the beam. The number of rivets necessary may be determined by assuming that the plate transmits to the floor beam that component of the stress in the inclined lateral members which acts in a direction at right angles to the axis of the chords.

Art. 17. Design of Bed Plates, Friction Rollers and Pedestals.

§ *100. All bed plates must be of such dimensions that the greatest pressure upon the masonry shall not exceed 250 pounds per sq. in.*

§ *101. All bridges over 75 ft. span shall have at one end nests of turned friction rollers running between planed surfaces. These rollers shall not be less than 2⅞ inches diameter for spans 100 feet or less, and for greater spans this diameter shall be increased in proportion of 1 inch for 100 feet additional.*

The rollers shall be so proportioned that the pressure per linear inch shall not exceed the product of the diameter in inches by 300 pounds.

§ *104. Pedestals shall be made of riveted plates and angles. All bearing surfaces of the base plates and vertical webs must be planed. The vertical webs must be secured to the web by angles having two rows of rivets in the vertical legs. No base plate or web connecting* *angle shall be less in thickness than $\frac{3}{4}$ inch. The vertical webs shall be of sufficient height, and must contain material and rivets enough to practically distribute the loads over the bearings or rollers.*

Where the size of the pedestal permits the vertical webs must be rigidly connected transversely.

§ *105. All the bed plates and bearings under fixed and movable ends must be fox bolted to the masonry; for trusses, these bolts must not be less than $1\frac{1}{4}$ inches diameter.*

The dead load reaction at the base plate is 325,000 pounds, and the live load 420,000 pounds; a total of 745,000 pounds. The masonry bearing will be $\frac{745,000}{250} = 2,980$ square inches. The diameter of the rollers according to the specification should be at least $2\frac{7}{8} + \frac{250}{100} = 5\frac{3}{8}$ inches, but since it is better practice to build segmental rollers the diameter of these rollers may be taken at 8 inches. Assuming then that eight rollers will furnish a proper form of distribution, the number of linear inches of length per roller required will be $\frac{745,000}{8 \times 300 \times 8} = 38.9$ inches; the rockers will therefore be 8 in number, 8 inches high, 5 inches wide and 4 feet 5 inches long, rolling on a base plate $1\frac{1}{4}$ inches thick, 4 feet 2 inches × 5 feet 6 inches in area. A $1\frac{1}{4}$-inch plate, 50 inches × 4 feet 6 inches, placed over the rockers will form the seat or bed plate for the pedestal proper. Both base plate and bed plate will have riveted to them, so placed as to be directly below the center of the end post, a flat bar, $2\frac{1}{4}$ inches × $\frac{3}{8}$ inch × 4 feet 2 inches long, made to fit corresponding grooves turned in the rockers to prevent all lateral motion of the rockers or pedestal. The rollers will be fastened rigidly together in their proper positions with $\frac{3}{8}$-inch clear spacing by a $\frac{3}{4}$-inch strap on each side 2 feet 3 inches long secured to the axis of each roller by a nut.

The pedestal must distribute uniformly to the shoe or bed plate the vertical load carried by the pin at the foot of the end post. For this reason it should be made deep enough to prevent appreciable deflection, but not so deep as to cause a tendency to buckle the side plates. A completely rational design for a pedestal is practically impossible. It is treated in some cases as a short plate girder uniformly supported along the bottom and carrying a concentrated load at its center. It is more usual to select some tried form of pedestal and adapt it for use in accordance with good judgment; this will be done in the present case. The side plates of the pedestal supporting the end pin must first be considered. The diameter of this pin should therefore be known, and in this instance may be assumed to be 7 inches. With an allowable bearing pressure of 15,000 pounds per square inch, the total thickness of the side plates for one half of one pedestal will be $745{,}000/2 \div (15{,}000 \times 7) = 3.55$ inches. If the main side plates are fastened between two $6 \times 6 \times \frac{3}{4}$ inch angles, $\frac{3}{4}$ filling plates will be required above the angles. In addition to these two $\frac{3}{4}$ inch plates there will then be needed two $\frac{11}{16}$ inch plates and one $\frac{3}{4}$ inch plate, giving a total thickness of $3\frac{5}{8}$ inches. An additional plate $\frac{7}{16}$ inch thick, called a lap plate, projecting upwards within the end post, will be fastened to the inside of these plates. The riveting of these connections is clearly shown in Plate II. It will be seen that a $\frac{7}{16}$-inch cover plate and two $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ inch angles are added to the upper edges of the pedestal in order to make it conform in finished appearance to the end post.

The pedestal at the fixed end of the truss will be exactly similar to the one above described, but the rollers will be dispensed with and the difference in height will be provided for in the pedestal itself.

The base plate, as shown in the figure, will be rigidly secured to the masonry by $1\frac{1}{2}$-inch anchor bolts 2 feet long. On the outer side of the rollers, a dust guard of two angles and a plate built in Z-shape will be connected to the anchor bolts.

ART. 18. DESIGN OF END FLOOR BEAM.

The end floor beam, which is fastened to the shoe plate and which carries not only the end stringers, but also the short cantilever brackets supporting the track from the abutment itself to the end stringer, still remains to be designed.

The end shear of this floor beam will be the live load shear of a single stringer, 62,500 pounds, together with an estimated dead load shear of 7,500 pounds, or a total of 70,000 pounds. The maximum moment will then be 70,000 pounds × 63 ins. = 4,440,000 inch pounds. Assuming the effective depth of girder to be 54 inches, the required sectional area of one flange will be 8.15 square inches and it may be composed of two angles $6 \times 4 \times \frac{1}{2}$ inch with a gross section of 9.6 square inches and a net section of 8.6 square inches.

The web will be a 55¾ × ⅜-inch plate with an area of 20.9 square inches. Allowing a compressive intensity stress of 8,000 pounds per square inch in the end stiffeners, the area required will be 8.75 square inches. Two 6×4×½-inch angles having an area of 9.5 square inches will be used, with fillers.

The end floor beam rests on and is riveted to a horizontal ½-inch plate, which in turn is solidly united to the shoe plate. To prevent lateral deflection, the upper flange of this floor beam is tied to the end post by means of a horizontal bent plate, and in addition a vertical triangular shaped plate is fastened to the end stiffeners and to the shoe plates, as shown clearly by Plate II. The plane of the lower lateral bracing does not coincide with the plane of the shoe plate, and to provide a proper connection, two 6 × 4 × ½-inch angles are riveted to the web of the floor beam at the proper elevation.

The end stringer bracket is also shown in sufficient detail in Plate II. The floor beam is slotted in order to allow the tie plate forming the cover plate of the bracket to pass through it. This tie plate is riveted to the upper flange of the adjacent intermediate stringer, to relieve as much as possible the direct stress of tension which may come upon the rivet heads connecting the bracket to the floor beam.

ART. 19. DESIGN OF PINS AND JOINT DETAILS.

§ 38. Pins shall be proportioned so that the shearing stress shall not exceed 9,000 pounds per square inch; nor the crushing stress on the projected area of the semi-intrados of any member (other than forged eye-bars, see § 80) connected to the pin, be greater per square inch than 15,000 pounds; nor the bending stress exceed 18,000 pounds, when the applied forces are considered as uniformly distributed over the middle half of the bearing of each member.

§ 78. The lower chord shall be packed as narrow as possible.

§ 80. The diameter of the pin shall not be less than ¾ the largest dimension of any eye-bar attached to it. The several members*

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*The following analysis furnishes the foundation for this specification. If the width of an eye-bar be represented by w, and the depth by h (the thickness of the head is taken the same as that of the bar) and if T be the permissible intensity of tensile stress, the total stress carried by the bar will be $w. h. T.$ If the diameter of the *attaching to the pin shall be so packed as to produce the least bending moment upon the pin, and all vacant spaces must be filled with wrought filling rings.*

§ 88. Where necessary, pin holes shall be reinforced by plates, some of which must be of the full width of the member, so that the allowed pressure on the pins shall not be exceeded, and so that the stresses shall be properly distributed over the full cross-section of the members. These reinforcing plates must contain enough rivets to transfer their proportion of the bearing pressure, and at least one plate on each side shall extend not less than 6 inches beyond the edge of the bottom plates (§ 87).

§ 87. The open sides of all compression members shall be stayed by batten plates at the ends. . . . The batten plates must be placed as near the ends as practicable, and shall have a length not less than the greatest width of the member or 1½ times its least width.

§ 89. Where the ends of compression members are forked to connect to the pins, the aggregate compressive strength of these forked ends must equal the compressive strength of the body of the members.

In addition to these direct specifications the following may be noted:

§ 60. In compression members, abutting joints with planed faces must be sufficiently spliced to maintain the parts accurately in contact against all tendencies to displacement.

§ 61. In compression members, abutting joints with untooled faces must be fully spliced, as no reliance will be placed on such abutting joints. . . .

The bending in pins and the allowed bearing of the plates on pins are peculiarly related; the larger the diameter of the pin, the narrower are the necessary bearing surfaces and consequently the smaller are the bending moments. If the diameter of the pin be decreased, the bearing surfaces must be increased in width. In that case the bending moments are increased and the pin diameter

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pin be represented by d and C be the permissible bearing intensity of stress on the pin, then $d. w. C$ will represent the bearing allowed on the pin. The two quantities $w. h. T$ and $d. w. C$ should be equal; if the ratio of C/T be taken at 4/3, then $whT = dwC$ and $d = 3/4h$.

may have to be increased. It will usually be impossible to obtain an economical balance for these conflicting conditions throughout the truss without many different sizes of pin. It is therefore customary to fix upon one size of pin for the lower chord, and another smaller one for the upper chord, then test them for bending and design the bearing surfaces to correspond with the diameters so chosen. The packing or arranging of the eye bars must be so chosen as to reduce the bending to a safe minimum.

Specification § 80 at once fixes a minimum limit for the size of the lower chord pins and also of the pins at the upper extremity of the end post to 6 inches; and the pins in the upper chord to $5\frac{1}{4}$ inches. The sizes tentatively assumed will be 7 inches for the first set of pins and 6 inches for the latter. The bearings of the various members for these sizes of pins are then designed, and then all the members at any one point so packed as to produce the least practicable bending moment. The "play" or spacing allowed in packing eye-bar heads may sometimes be taken as small as $\frac{1}{32}$ inch to allow for slight imperfections in manufacture, but it is usually taken as $\frac{1}{8}$ inch for each eye-bar head. This amount may be increased to meet the requirements of any special case. The clear distance between built members such as chords or posts is usually greater; it may be taken as high as 1 inch. It will be found necessary at some connecting points to cut away parts of the flanges of channels or angles in built-up members on account of the interference of inclined members. If this must be done, the remaining part of the member must always be tested by computation as to its strength, considering it as a short piece in compression. In most cases the usual pin bearing plates are sufficient to replace the metal cut away. The following formula * has been much used for the purpose of designing such forked portions of the ends of posts:

$$t = \frac{P}{8000b} + \frac{l}{27}, \qquad \text{Eq. (1)}$$

in which t represents the total thickness of metal whose width is b; P the total load on one jaw of the post (usually one half the total load carried by the post or column) and l the distance from the center of the pin hole to the last center line of rivets in the body of the column back of the cut in the angle or in the flange of the channel. This formula is applicable to steel with ultimate

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[1] See Burr's "Resistance of Materials," page 515.

tensile resistance running from 60,000 to 68,000 pounds per square inch. For higher steel, or for highway bridges, or for other structures where less margin of safety may be justifiable, the value of t may be made correspondingly less than given by Eq. (1).

Pin Plates in Posts.

The posts at their lower extremities carry to the pins not only their maximum stresses as members of the truss but also the compression due to their acting as supporting struts for the floor beams. It is possible and it should be assumed that the maximum post stress and the greatest panel load occur together. In the present case this requires that the lower ends of the posts be reinforced with sufficient thickness of bearing plates to carry 98,000 pounds in addition to the stress taken from the stress sheet (Plate I.). In order that this floor beam load may be distributed uniformly to the pin by both jaws of the column, it is necessary that the two sides of the post be riveted firmly together to act as a whole; otherwise the inside jaw will carry the greater portion of the load. This is usually accomplished by inserting between the two parts of the column, at the end of the floor beam, a diaphragm which acts as a continuation of the web plate of the beam. This diaphragm will be composed of a $\frac{3}{8}$-inch plate riveted between four 4 × 3 × $\frac{3}{8}$-inch angles. The method of fastening is clearly shown in Plate II.

Member L_1U_1 — Lower End.

This member needs to be fastened at its lower end only to increase the general stiffness of the structure, since it is simply a hanger to carry load to the upper panel point. The *U*-shaped plate to which the lower lateral bracing is attached furnishes inside pin plates, but in addition two $\frac{3}{8}$-inch plates will be fastened to the outside extending upwards to the bottom of the floor beam, so that the latter may have support during erection.

Member L_1U_1 — Upper End.

A 1-inch plate, bearing on a 7-inch pin, may carry 105,000 pounds. The load to be carried at this point is 117,000 pounds; therefore 1.12 inch total width of bearing will be required. As the webs of the channel are each 0.375 inch thick the thickness of pin plate required would be 0.37 inch, but since the net section of the channels is reduced by the diameter of the pin, it will be advisable to use two 5-inch pin plates, especially as the vertical distance from the top of the pin to the upper extremity of the channels is but 6 inches. The net section through this point should not be less than $\frac{2}{3}$ of the transverse net section, and the $\frac{5}{8}$-inch pin plates chosen will satisfy the condition.

Since the allowable stress in the main section is 6,000 pounds per sq. in., each pin plate may be supposed to carry $15 \times \frac{5}{8} \times 6{,}000 = 56{,}300$ pounds, and as the rivet bearing in a $\frac{8}{8}$-inch plate is 4,920 pounds, 12 rivets will be required.

Member U_2L_2 *— Lower End.*

The sum of the dead and live loads carried at this point, together with the floor beam load of 98,000 pounds, may produce a total load of 357,000 pounds, or 178,500 pounds on each jaw of the post resting on a 7-inch pin. A total thickness of $\frac{178{,}500}{105{,}000} = 1.7$ inches will therefore be required. A $\frac{5}{8}$-inch inside, and a $\frac{5}{8}$-inch outside pin plate, together with the $\frac{9}{16}$-inch main plate, give a thickness of 1.81 inches, and will prove satisfactory.

Since the bearing value for one rivet in a $\frac{9}{16}$-inch plate is 7,380 pounds, or less than the double shear value, the former quantity will determine the number of rivets. It may be assumed that the total stresses carried by the different plates are proportional to their thickness; therefore $(\frac{5}{8} + \frac{5}{8}) \div 1\frac{9}{16} = 69\%$ of the total stress must be transferred by rivets from the main plate to the pin plates. In this case $0.69 \times 178{,}500 = 123{,}000$ pounds; therefore at least 17 rivets will be required; Plate II. shows the arrangement of plates and rivets.

Member U_2L_2 *— Upper End.*

The 6-inch pin at this point will carry a bearing pressure of 90,000 pounds per inch width of plate, the load on each jaw of this post being 129,500 pounds. 1.44 inches thickness of bearing will be required, and this thickness will be afforded by the $\frac{9}{16}$-inch main plate, with a $\frac{7}{16}$-inch outside and a $\frac{1}{2}$-inch inside pin plate. The bearing value of one rivet in a $\frac{9}{16}$-inch plate is 7,380 pounds, and the rivets transfer 63 per cent. of 129,500 = 81,500 pounds, to the pin plates, hence the required number will be 11. Plate II. shows the arrangement adopted.

Member U_3L_3 *— Lower End.*

The load carried by one jaw resting on the 7-inch pin, including the floor beam load, will be 101,500 pounds and the thickness required will be $\frac{101500}{105000} = 0.97$ inch. A ⅝-inch pin plate in addition to the ⅜-inch web of the channel will therefore suffice. The stress transferred to the pin plate will be 63,500 pounds and as the bearing value of one rivet in a ⅜-inch plate is 4,920 pounds, 13 rivets will be required.

Member U_8L_8 — *Upper End.*

The thickness of plates for one jaw bearing on a 6-inch pin will be $\frac{52500}{90000} = 0.58$ inches. A ½-inch pin plate will be riveted to the outside of the channel and the number of rivets required will be 6.

It will not be necessary to repeat these operations in detail for the other posts of this truss, the method of design being precisely the same as those already given.

ART. 20. DESIGN OF PIN CONNECTIONS OF UPPER CHORD MEMBERS.

As the joint of two adjoining upper chord members having different inclinations is always at the angle in the chord with its plane passing through the axis of the pin, it is usually made abutting and riveted so as to make the chord continuous. This makes the chord stiffer than if the joints were pin bearing. It has become almost the invariable practice, however, to make the joint at the top of the end post an open pin bearing one. Since all abutting faces will be planed, specification § 60 requires no splice plates except those sufficient to prevent displacement. The only stress transferred from a pin to a chord member will then be the resultant, in the direction of the chord member, of the maximum stresses in the web members meeting at that point, *i. e.*, the increment of stress. Sufficient pin-bearing area must be provided to carry this increment of stress.

The joint for adjoining chord members whose axes are in the same straight line is never made at a pin, but at some distance from it, on that side away from the center. This procedure is adopted for convenience in erection which usually proceeds from the center towards the ends. This joint is placed as near the pin as practicable, and in a chord 20 inches deep, it is usual to have three rows of rivets in the splice plate on each side of the joint; for splices in chords 24 to 30 inches deep 4 rows are usually inserted.

At such a joint the batten plate on the bottom and a short cover plate on top are used to strengthen the splice.

It has already been noted that the axes of the pins have been placed $14\frac{1}{2}$ inches below the edge of the cover plate, or 0.7 inch below the center of gravity of the chord section. This tends in some degree to neutralize the flexure due to own weight by the counter flexure introduced by the increment of chord stress.

In the bridge under consideration no joint will be made at panel points U_4 and U_6; the members $U_5U_4U_5$ and $U_5U_6U_5$ will each be made in single pieces about 59 feet long, which is entirely practicable. The supporting struts which frame into these members at their center points are connected by means of angles, as shown in Plate II. The joint for U_2 is made 2 feet $4\frac{1}{2}$ inches away from the center of U_2 and $\frac{7}{16}$-inch splice plates will be riveted on the inside of the side plates of the chord, with 4 rows of rivets on each side of the joint. A $\frac{1}{8}$-inch filler is necessary to allow for the difference in thickness of plates in the two chord members. The $\frac{1}{2}$-inch outside plate will be continued to cover the joint.

The maximum increment of stress in U_2U_3 given by the web member at U_2 is 140,000 pounds, or 70,000 pounds for each jaw, The 6-inch pin can transfer this stress to the $\frac{18}{16}$-inch side plate, but for increased stiffness and solidity a $\frac{1}{2}$-inch plate with $\frac{1}{2}$-inch filler will be riveted on the outside of each jaw. As may be observed, all connections are made so strong that a member should fail rather in its body than at its end connections.

Connection at U_3.

The maximum increment of stress for the member U_3U_4 at panel U_3 in the direction of the axis of U_3U_4 is found graphically from the stresses in the web members to be 200,000 pounds. The thickness of each jaw bearing on a 6-inch pin must then be at least 1.1 inches. It will be seen in Fig. 7 that besides the $\frac{9}{16}$-inch filler plate and the $\frac{3}{8}$-inch lap plate, a $\frac{3}{8}$-inch inside and a $\frac{3}{8}$-inch outside pin plate have been used. This is more than the computations require, but not more than desirable for the stiffness of the continuous upper chord.

Plates corresponding to those riveted to U_3U_4 are fastened to U_2U_3 at U_3. The outside edges of the main side plates of adjacent members should always be in line. In this truss the uniform distance from back to back of these plates is 23 inches. The abutting pieces at U_3 are shown in Fig. 7.

The stress carried by each plate will be determined by multiplying its thickness in inches by the allowed bearing of a 1-inch plate on a 6-inch pin, and this quantity determines the number of rivets required in each plate. These rivets must be designed not only for single shear for each separate plate, but also for the combined bearing of the outside plates on the main inside web plate. Where field rivets are used their stress value is only two thirds that of shop rivets.

If the thickness of pin plates is large, considerable bending, indeterminate in amount, may take place in the rivets, and it is proper on this account to add more rivets than those actually required by computation. Moreover, since the cover plate and top angles of the chord section receive their stress cumulatively, the pin plates which transfer a part of the stress to them should be made long enough to enable this stress to be transferred directly, instead of first passing through the main web plate.

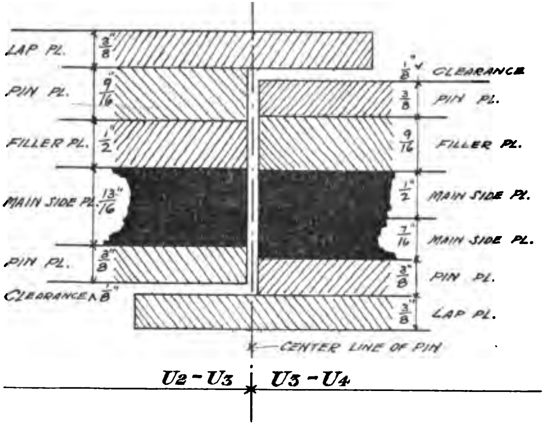


FIG. 7.

Since there is thus a certain amount of indetermination in the transmission of stress in an abutting joint, such a joint is not designed strictly according to computations, but more or less according to experience tempered by the engineer's judgment. The design adopted must, however, always be tested by some method of analysis to determine its safety.

Connection at U_5.

The pin plates at this panel point are designed in a manner similar to that followed for U_3. The design adopted, Plate II., provides abundant material.

ART. 21. DESIGN OF DETAILS AT ENDS OF END POST.

Lower Extremity.

The end connections of L_0U_1 are not abutting, but pin bearing. Hence all the stress in L_0U_1, 899,000 pounds, is transferred to the pin, and from that to the pedestal plates.

A 1-inch plate bearing on a 7-inch pin can carry 7 × 15,000 = 105,000 pounds; therefore a 4.28-inch thickness of pin plate is required for each jaw. There will then be added to the main web plates $\frac{1}{2}$ inch and $\frac{7}{16}$ inch thick respectively, an outside $\frac{5}{8}$-inch lap plate, a $\frac{5}{8}$-inch plate below this and a $\frac{9}{16}$-inch plate over a $\frac{1}{2}$-inch filler. On the inside, next to the side plate, there will be a $\frac{9}{16}$-inch plate and then a $\frac{7}{16}$-inch plate; this gives a bearing thickness of $4\frac{1}{2}$ inches. The following rivets will be required in single shear for the various plates; Plate II. shows their distribution:

Outside Plates.		Inside Plates.	
$\frac{5}{8}$-inch lap plate....................	12 rivets.	$\frac{7}{16}$-inch plate........................	9 rivets
$\frac{5}{8}$-inch plate........................	12 "		
$\frac{9}{16}$-inch "	11 "	$\frac{9}{16}$-inch plate........................	11 "
$\frac{1}{2}$-inch "	10 "		

Upper Extremity.

The same thickness of $4\frac{1}{4}$ inches of pin plate is required at the upper extremity as at the lower, but a portion of this thickness will be formed of an inside diaphragm, partly to avoid the inside pin plates interfering with the web members, and partly for the sake of the additional stiffness gained by this form of connection.

The diaphragm will be composed of a $\frac{11}{16}$-inch web, with four 4 × $3\frac{1}{2}$ × $\frac{1}{2}$-inch angles built in the form of a vertical I. Two $\frac{1}{2}$-inch fillers and two $\frac{9}{16}$-inch plates will be added to the diaphragm to bear on the pin. Fig. 8 shows clearly the dimensions of the pin plates fastened to the side plates. The computations need not be repeated here, it being remembered that one half the diaphragm plate thickness should be included in the thickness of one jaw.

End Connection of U_1U_2.

The connection of U_1U_2 at U_1 is designed in precisely the same manner as the end post connection at U_1. The stress in one jaw is 429,000 pounds, and 4.1 inches is the thickness of pin plates required. An inside diaphragm will again be used and Fig. 8 shows clearly the sections adopted.

Since the end post and U_1U_2 are not riveted together it is usual to allow a small clearance between them for ease in erection. Fig. 8 shows the clearance adopted.

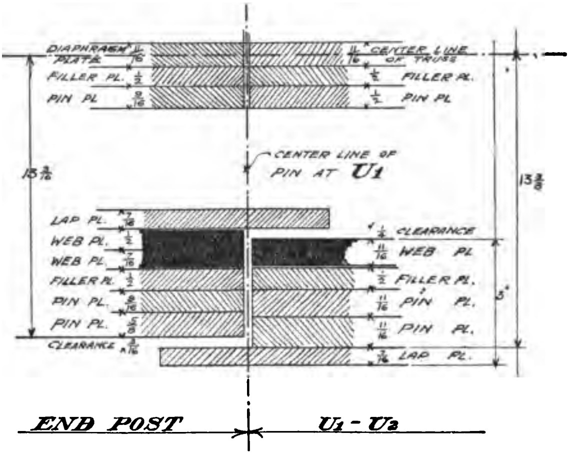


FIG. 8.

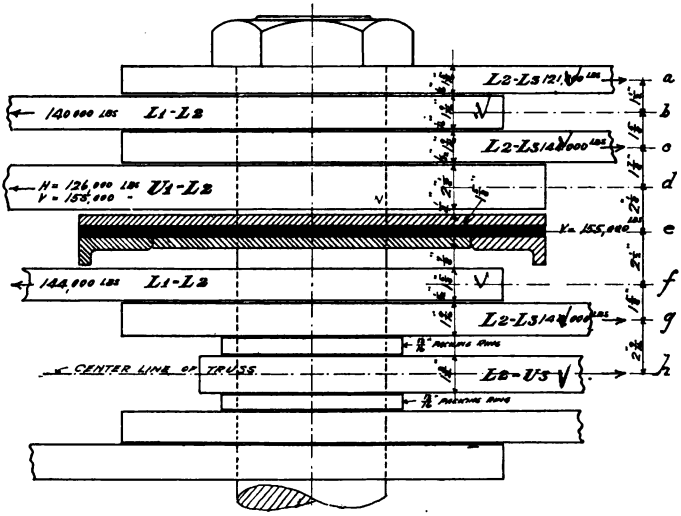
It will be seen that open joints are subject to a better defined analysis than abutting joints, but they form a heavier, more cumbersome and less rigid connection.

ART. 22. BENDING OF PINS.

Lower Chord Packing.

Panel Point L_2.

The packing of pin L_2 and the computations of the bending moments in it will furnish characteristic operations typical of all other cases. Fig. 9 shows the disposition of the members meeting at L_2, the clearances, and distances between centers of adjacent pieces and the stresses carried by the pieces when L_1L has its maximum stress. The maximum stresses in all the members meeting at one point are not concurrent since they are caused by different positions of the moving load. It becomes necessary to test each pin, (1) with the maximum stresses existing in the web members, and (2) with the maximum stresses existing in the chord members. (It is sufficiently accurate to assume that the same position of loading causes the maximum stresses in the two adjacent chord members or in the web members converging on the pin.) As a rule, the greatest pin bending in the lower chord occurs with the greatest chord stresses, and in the upper chord with the greatest web stresses.



7" PIN ~ L2

FIG. 9.

The wind load stresses in the lower chord may be neglected as a factor in producing bending in the pins.

Taking moments about the center of each piece in turn there is found:

	Horizontal Moment.	Vertical Moment.
About *b*	+ 181,500 in. lbs.	
" *c*	+ 151,000 " "	
" *d*	+ 385,000 " "	
" *e*	+ 383,000 " "	330,000 in. lbs.
" *f*	+ 380,000 " "	330,000 " "
" *g*	+ 144,000 " "	

The maximum moment, therefore, occurs at *e* and is equal to

$$\sqrt{383{,}000^2 + 330{,}000^2} = 505{,}000 \text{ inch pounds.}$$

The allowed unit bending stress is 18,000 pounds per sq. in., and the bending resistance of a 7-inch pin is 606,100 inch pounds The packing at this point is, therefore, satisfactory.

If the bending be computed when L_2U_1 has its maximum stress, the vertical component of which is 166,000 pounds, it is seen that the vertical moment would be increased slightly, but the horizontal moment would be more than correspondingly decreased. The maximum bending moment on the pin occurs in this case, therefore, with the maximum chord stress.

Other panel points in the lower chord are treated in precisely the same manner; Plate II. shows the packing employed.

Upper Chord Packing.

The packing at panel point U_3 may be assumed characteristic of the entire upper chord; Fig. 10 shows clearly the spacing and also the stresses in the different parts, when U_3M_4 takes its maximum stress. Since the two chord members U_2U_3 and U_3U_4 are abutting and as there is practically no eccentricity between them, the resultant stress in the web members U_3L_3 and U_3M_4 is entirely resisted by U_3U_4, the horizontal and vertical components of which are easily found graphically.

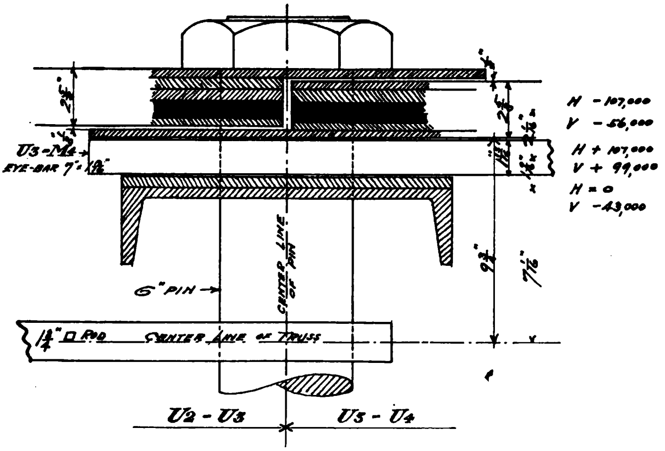


FIG. 10.

The maximum stress in U_3M_4 is + 294,000 pounds, and the concurrent stress in U_3L_3 is – 86,000 pounds.

The side plates of the chord form the supports of the ends of the pin, the center of the supporting forces being at the center of the plates, as shown in Fig. 10. The members causing the bending in the pin are U_3M_4 and U_3L_3, the stress in the latter, – 86,000 pounds, being wholly vertical. The horizontal and vertical components of the stress in one half of U_8M_4 are :

$$\begin{aligned} H. &= +\ 107{,}000 \text{ lbs.} \\ V. &= +\ 99{,}000 \text{ ``} \end{aligned}$$

Fig. 10 shows that the bending moment in the central portion of the pin is then composed of

$$\begin{aligned} &\text{Hor. Mom.} = 107{,}000 \times 2\tfrac{1}{16} = 221{,}000 \text{ inch pounds.} \\ &\text{Vert. `` } = 99{,}000 \times 2\tfrac{1}{16} - 43{,}000 \times 3\tfrac{5}{8} = 48{,}000 \text{ inch pounds.} \end{aligned}$$

Hence the resultant moment is:

$$\sqrt{221{,}000^2 + 48{,}000^2} = 226{,}000 \text{ inch pounds.}$$

The allowed bending resistance of a 6-inch pin is 318,000 inch pounds. A 6-inch pin is therefore satisfactory at U_8.

Other upper chord panel points are investigated in exactly the same manner. It is desirable to repeat the operations in detail for that panel point only at the upper extremity of the end posts (Fig. 11), and this point will be investigated for that position of loading which causes the maximum stress in U_1L_2 and the following table shows the components of the direct stresses in the members meeting at U_1 for that condition of loading:

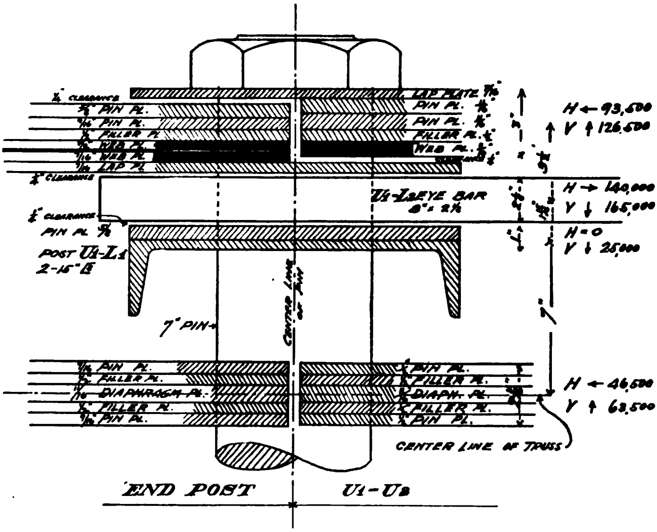


FIG. 11.

	Vertical Components.	Horizontal Components.
U_1L_1	+ 50,000 pounds.	——
L_0U_1	—620,000 “	—510,000 pounds.
U_1U_2	+240,000 “	+790,000 “
U_1L_2	+330,000 “	—280,000 “

No essential error will be introduced if it be assumed that there is no eccentricity in the line of action of L_0U_1 (the end-post) and U_1U_2. The resulting differences between their stresses may therefore be taken exactly along their center lines; a part of this stress is carried by the diaphragm, in this case about 34 per cent. Fig. 11 shows the stresses carried at one half of the panel point.

The resultant moments in inch pounds are as follows:

Center of Moments at Center of	Horizontal Moment.	Vertical Moment.	Maximum Moment.
U_1L_2	← 306,000	↑ 415,000	515,000
U_1L_1	← 222,500	↑ 345,000	
center line of truss.	→ 103,500	↓ 99,500	

The allowed bending resistance of a 7-inch pin is 606, 100 inch-pounds; and therefore satisfies the requirements at U_1.

ART. 23. CAMBER.

§ *107. All bridges shall be cambered by giving the panels of the top chord an excess of length in the proportion of ⅛ inch to every 10 feet.*

In accordance with this specification the following table shows the original lengths and the changed lengths of the upper chord members. It is seen that the end post is not changed in length.

	Original Length.	Changed Length.
L_0U_1	46′ 4″	46′ 4″
U_1U_2	30′ 6 9/32″	30′ 6⅝″
U_2U_3	“	“
U_3U_4	29′ 6⅛″	29′ 6½″
U_4U_5	“	“
U_5U_6	29′ 2″	29′ 2⅜″

No changes are made in the lengths of the main web members, their final lengths being the same as if no changes had been made in the lengths of the upper chord members.

The stretching of the lower chord panels makes it necessary to increase the lengths of M_3M_4 and M_5M_6 to an amount equal to half that stretch. The final length of each of these members will then be 29 feet $2\frac{3}{16}$ inches.

ART. 24. CONCLUSION.

The principles outlined indicate a system of rational treatment for the design of all parts of a modern truss bridge. It is obvious that a wide range of treatment of details is permissible in securing the above results, and those given here are subject to this general observation. Other details of equal efficiency might be designed but those given are at least as effective as others which might have been used and have served the purpose of illustration at least as well.

A carefully made bill of materials with the resulting computation of weights would show that the total shipping weight of this 350-ft. single track through span would be 821,000 pounds.

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